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Abbreviations Used in this Section

AASHO	-	American Association of State Highway Officials (1921 to 1973)
AASHTO	-	American Association of State Highway and Transportation Officials (1973 to present)
AASHTO Manual	-	*Manual for Maintenance Inspection of Bridges*
BIRM	-	*Bridge Inspector's Reference Manual*
BMS	-	Bridge Management System
Coding Guide	-	FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*
DOT	-	Department of Transportation
FCM	-	fracture critical member
FHWA	-	Federal Highway Administration
HBRR	-	Highway Bridge Replacement & Rehabilitation
HEC	-	*Hydraulic Engineering Circular*
ISTEA	-	Intermodal Surface Transportation Efficiency Act
Manual 70	-	*Bridge Inspector's Training Manual 70*
Manual 90	-	*Bridge Inspector's Training Manual 90*
MR&R	-	maintenance, repair and rehabilitation
NBI	-	National Bridge Inventory
NBIS	-	National Bridge Inspection Standards
NCHRP	-	National Cooperative Highway Research Program
NDT	-	nondestructive testing
NHI	-	National Highway Institute
NHS	-	National Highway System
NICET	-	National Institute for Certification in Engineering Technologies
TEA-21	-	Transportation Equity Act of the 21st Century
TRB	-	Transportation Research Board
TWG	-	Technical Working Group

Section 1

Bridge Inspection Programs

Topic 1.1 History of the National Bridge Inventory Program

1.1.1 Introduction

In the years since the Federal Highway Administration's landmark publication, *Bridge Inspector's Training Manual 90* (*Manual 90*), bridge inspection and inventory programs of state and local governments have formed an important basis for formal bridge management programs. During the 1990's, the state DOT's implemented comprehensive bridge management systems, which rely heavily on accurate, consistent bridge inspection data.

This manual (*Bridge Inspector's Reference Manual*) updates *Manual 90* and reflects over ten years of change.

Advances in technology and construction have greatly enhanced current bridge design. However, the emergence of previously unknown problem areas and the escalating cost of replacing older bridges make it imperative that existing bridges be evaluated properly to be kept open and safe.

There are four letters that define the scope of bridge inspections in this country: NBIS, meaning National Bridge Inspection Standards. The **National Bridge Inspection Standards (NBIS)** are Federal regulations establishing requirements for:

- Inspection Procedures
- Frequency of Inspections
- Qualifications of Personnel
- Inspection Reports
- Maintenance of Bridge Inventory

The **National Bridge Inventory (NBI)** is the aggregation of structure inventory and appraisal data collected by each state to fulfill the requirements of NBIS.

To better understand the National Bridge Inventory Program, it is helpful if we take a look back at the development of the program.

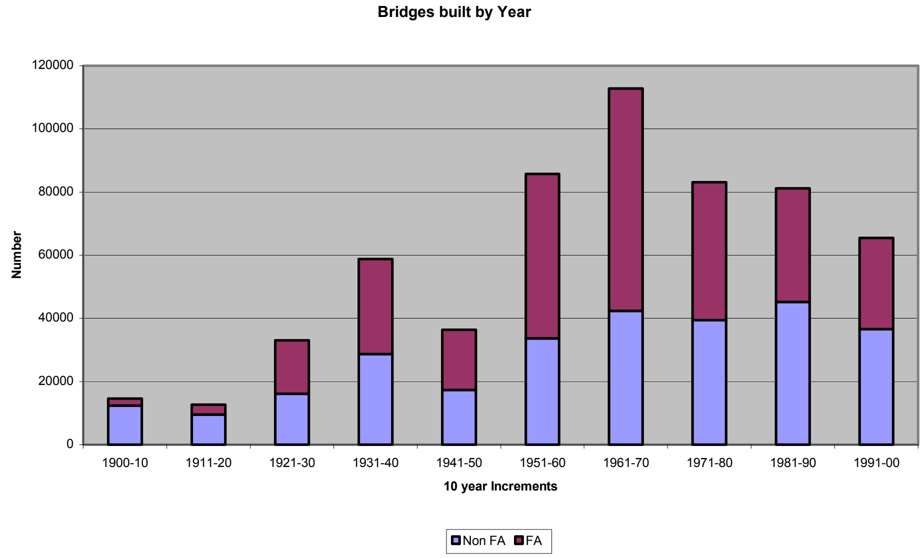

Table 1.1.1 Number of Bridges Built since 1900

1.1.2 History of the National Bridge Inventory Program

Background

During the bridge construction boom of the 1950's and 1960's, little emphasis was placed on safety inspection and maintenance of bridges. This changed when the 681 m (2,235-foot) Silver Bridge, at Point Pleasant, West Virginia, collapsed into the Ohio River on December 15, 1967, killing 46 people (see Figure 1.1.1).

Figure 1.1.1 Collapse of the Silver Bridge

This tragic collapse aroused national interest in the safety inspection and maintenance of bridges. The U.S. Congress was prompted to add a section to the "Federal Highway Act of 1968" which required the Secretary of Transportation to establish a national bridge inspection standard. The Secretary was also required to develop a program to train bridge inspectors.

The 1970's

Thus, in 1971, the National Bridge Inspection Standards (NBIS) came into being. The NBIS established national policy regarding:

- Inspection procedures
- Frequency of inspections
- Qualifications of personnel
- Inspection reports
- Maintenance of state bridge inventory (NBI)

Three manuals were subsequently developed. These manuals were vital to the early success of the NBIS. The first manual was the Federal Highway Administration (FHWA) *Bridge Inspector's Training Manual 70* (*Manual 70)*. This manual set the standard for inspector training.

The second manual was the American Association of State Highway Officials (AASHO) *Manual for Maintenance Inspection of Bridges*, released in 1970. This manual served as a standard to provide uniformity in the procedures and policies for determining the physical condition, maintenance needs and load capacity of highway bridges.

The third manual was the FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (*Coding Guide*), released in July 1972. It provided thorough and detailed guidance in evaluating and coding specific bridge data.

With the publication of *Manual 70*, the implementation of national standards and guidelines, the support of AASHO, and a newly available FHWA bridge inspector's training course for use in individual states, improved inventory and appraisal of the nation's bridges seemed inevitable. Several states began in-house training programs, and the 1970's looked promising. Maintenance and inspection problems associated with movable bridges were also addressed. In 1977, a supplement to *Manual 70*, the *Bridge Inspector's Manual for Movable Bridges*, was added.

However, the future was not to be trouble free. Two predominant concerns were identified during this period. One concern was that bridge repair and replacement needs far exceeded available funding. The other was that NBIS activity was limited to bridges on the Federal Aid highway systems. This resulted in little incentive for inspection and inventory of bridges not on Federal Aid highway systems.

These two concerns were addressed in the "Surface Transportation Assistance Act of 1978." This act provided badly needed funding for rehabilitation and new construction and required that all public bridges over 20 feet (6.1 m) in length be inspected and inventoried in accordance with the NBIS by December 31, 1980.

Any bridge not inspected and inventoried in compliance with NBIS would be ineligible for funding from the special replacement program.

In 1978, the American Association of State Highway and Transportation Officials (AASHTO) revised their *Manual for Maintenance Inspection of Bridges* (*AASHTO Manual*). In 1979, the NBIS and the FHWA *Coding Guide* were also revised. These publications, along with *Manual 70*, provided state agencies with definite guidelines for compliance with the NBIS.

The 1980's

The National Bridge Inspection Program was now maturing and well positioned for the coming decade. Two additional supplements to *Manual 70* were published. First, culverts became an area of interest after several tragic failures. The 1979 NBIS revisions also prompted increased interest in culverts. The *Culvert Inspection Manual* was published July 1986. Then, an emerging national emphasis on fatigue and fracture critical bridges was sharply focused by the collapse of Connecticut's Mianus River Bridge in June 1983. *Inspection of Fracture Critical Bridge Members* was published in September 1986. These manuals were the products of ongoing research in these problem areas.

With the April 1987 collapse of New York's Schoharie Creek Bridge, national attention turned to underwater inspection. Of the 592,000 bridges in the national inventory, approximately 86% are over waterways. The FHWA responded with *Scour at Bridges*, a technical advisory published in September 1988. This advisory provided guidance for developing and implementing a scour evaluation program for the:

- Design of new bridges to resist damage resulting from scour
- Evaluation of existing bridges for vulnerability to scour
- Use of scour countermeasures
- Improvement of the state-of-practice of estimating scour at bridges

Further documentation is available on this topic in the *Hydraulic Engineering Circular No. 18* (*HEC-18*).

In September 1988, the NBIS was modified, based on suggestions made in the "1987 Surface Transportation and Uniform Relocation Assistance Act," to require states to identify bridges with fracture critical details and establish special inspection procedures. The same requirements were made for bridges requiring underwater inspections. The NBIS revisions also allowed for adjustments in the frequency of inspections and the acceptance of National Institute for Certification in Engineering Technologies (NICET) Level III and IV certification for inspector qualifications.

In December 1988, the FHWA issued a revision to the *Coding Guide*. This time the revision would be one of major proportions, shaping the National Bridge Inspection Program for the next decade. The *Coding Guide* provided inspectors with additional direction in performing uniform and accurate bridge inspections.

The 1990's

The 1990's was the decade for bridge management systems (BMS). Several states, including New York, Pennsylvania, North Carolina, Alabama and Indiana, had their own comprehensive bridge management systems.

In 1991, the FHWA sponsored the development of a bridge management system called "Pontis" which is derived from the Latin word for bridge. The Pontis system has sufficient flexibility to allow customization to any agency or organization responsible for maintaining a network of bridges.

Simultaneously, the National Cooperative Highway Research Program (NCHRP) of the Transportation Research Board (TRB) developed a BMS software called "Bridgit." Bridgit is primarily targeted to smaller bridge inventories or local highway systems.

As more and more bridge needs were identified, it became evident that needed funding for bridge maintenance, repair and rehabilitation (MR&R) far exceeded the available funding from federal and state sources. Even with the infusion of financial support provided by the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, funding for bridge MR&R projects was difficult to obtain. This was due in part to the enormous demand from across the nation. An October 1993 revision to NBIS permitted bridge owners to request approval from FHWA of extended inspection cycles of up to four years for bridges meeting certain requirements.

In 1994, the American Association of State Highway and Transportation Officials (AASHTO) revised their *Manual for Condition Evaluation of Bridges* (*AASHTO Manual*). In 1995, the FHWA *Coding Guide* was also revised. These publications, along with *Manual 90, Revised July 1995*, provided state agencies with continued definite guidelines for compliance with the NBIS and conducting bridge inspection.

Although later rescinded in the next transportation bill, the ISTEA legislation required that each state implement a comprehensive bridge management system by October 1995. This deadline represented a remarkable challenge since few states had previously implemented a BMS that could be considered to meet the definition of a comprehensive BMS. In fact, prior to the late 1980's, there were no existing management systems adaptable to the management of bridge programs nor was there any clear, accepted definition of key bridge management principles or objectives.

This flexibility in the system was the result of developmental input by a Technical Working Group (TWG) comprised of representatives from the FHWA, the Transportation Research Board (TRB) and the following six states: California, Minnesota, North Carolina, Tennessee, Vermont and Washington. The TWG provided guidance drawing on considerable experience in bridge management and engineering.

The National Highway System (NHS) Act of 1995 rescinded the requirement for bridge management systems. However, many of the states continued to implement the Pontis BMS.

The Transportation Equity Act of the 21st Century (TEA-21) was signed into law in June 1998. TEA-21 builds on and improves the initiatives established in ISTEA and, as mentioned earlier, rescinded the mandatory BMS requirement.

The 2000's

In 2002, *Manual 90* was revised and updated as a part of a complete overhaul of the FHWA Bridge Safety Inspection training program. The new manual was named the *Bridge Inspector's Reference Manual* (BIRM) and incorporated all of *Manual 90.* The BIRM also incorporates manual 70 Supplements for culvert inspection and Fracture Critical Members, and course curriculum material that was not specifically part of the revised course objectives. Over the years, varying amounts of federal funds have been spent on bridge projects, depending on the demands of the transportation infrastructure. Table 1.1.2 illustrates the fluctuations in federal spending and shows current trends.

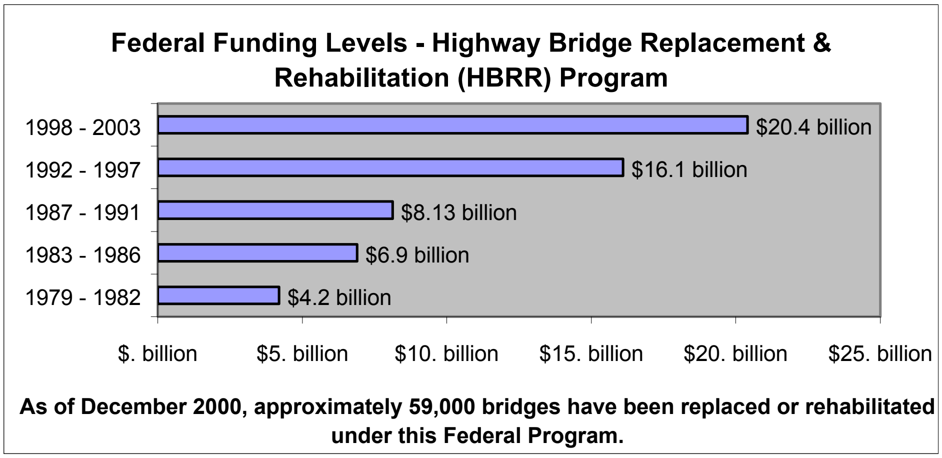

Table 1.1.2 Federal Funding Levels (1979 – 2003)

1.1.3 Today's National Bridge Inspection Program

Much has been learned in the field of bridge inspection, and a national Bridge Inspection Training Program is now fully implemented. State and federal inspection efforts are more organized, better managed and much broader in scope. The technology used to inspect and evaluate bridge members and bridge materials has significantly improved.

Areas of emphasis in bridge inspection programs are changing and expanding as new problems become apparent, as newer bridge types become more common, and as these newer bridges age enough to have areas of concern. Guidelines for inspection ratings have been refined to increase uniformity and consistency of inspections. Data from bridge inspections has become critical input into a variety of analyses and decisions by state agencies and the Federal Highway Administration.

The NBIS has kept current with the field of bridge inspection. The 1995 National Bridge Inspection Standards appear in Appendix A. The standards are divided into the following sections:

- Application of standards
- Inspection procedures
- Frequency of inspections
- Qualifications of personnel

- Inspection report
- Inventory

The FHWA has made a considerable effort to make available to the nation's bridge inspectors the information and knowledge necessary to accurately and thoroughly inspect and evaluate the nation's bridges.

FHWA Training

The FHWA has developed and now offers the following training courses relative to bridge inspection through the National Highway Institute (NHI):

- "Bridge Inspector's Training Course, Part I - Engineering Concepts for Bridge Inspectors" (NHI Course Number 130054)

  This one-week course presents engineering concepts, as well as inspection procedures and information about bridge types, bridge components, and bridge materials. The one-week course is for new inspectors with little or no practical bridge inspection experience.

- "Bridge Inspector's Training Course, Part II - Safety Inspection of In-Service Bridges" (NHI Course Number 130055)

  This two-week course is for experienced inspectors or engineers who perform or manage bridge inspections. Emphasis is on inspection applications and procedures. The uniform coding and rating of bridge elements and components is also an objective of the two-week course. A unique feature of this course allows for customization of the course content by the host agency. Some states use component rating based on NBIS while some states use element condition level based on Pontis. Optional topics can be scheduled, and their level of coverage can be selected. These topics include identification and inspection of fracture critical members (FCM's), underwater inspection, culverts, field trips, case studies, movable bridges, and coatings. Several special bridge types may also be discussed at the host agency's request.

- "Fracture Critical Inspection Techniques for Steel Bridges" (NHI Course Number 130078)

  This three and one-half day course provides an understanding of fracture critical members (FCM's), FCM identification, failure mechanics and fatigue in metal. Emphasis is placed on inspection procedures and reporting of common FCM's and nondestructive testing (NDT) methods most often associated with steel highway bridges.

- "Bridge Inspection Refresher Training" (NHI Course Number 130053)

  This three-day course provides a review of the National Bridge Inventory (NBI) inspection methods and includes discussions on structure inventory items, structure types, and the appropriate codes for the Federal Structure, Inventory and Appraisal reporting.

- "Stream Stability and Scour at Highway Bridges for Bridge Inspectors"

(NHI Course Number 135047)

This one-day course concentrates on visual keys to detecting scour and stream instability problems. The course emphasizes inspection guidelines to complete the hydraulic and scour-related coding requirements of the National Bridge Inspection Standards (NBIS).

- "Bridge Coatings Inspection" (NHI Course Number 130079)

  This four-day course provides information on the inspection of surface preparation and application of protective coating systems for bridge and highway structures. The course provides a basic overview of the theory of corrosion and its control and the characteristics of various bridge coating types.

Throughout all the expansions and improvements in bridge inspection programs and capabilities, one factor remains constant: the overriding importance of the inspector's ability to effectively inspect bridge components and materials and to make sound evaluations with accurate ratings. The validity of all analyses and decisions based on the inspection data is dependent on the quality and the reliability of the data collected in the field.

Across the nation, the duties, responsibilities, and qualifications of bridge inspectors vary widely. The two keys to a knowledgeable, effective inspection are training and experience in performing actual bridge inspections. Training of bridge inspectors has been, and will continue to be, an active process within state highway agencies for many years. This manual is designed to be an integral part of that training process.

Current FHWA Reference Material

- NBIS. *Code of Federal Regulations.* 23 Highways Part 650, Subpart C – National Bridge Inspection Standards.

- AASHTO. *LRFD Bridge Design Specifications, 2nd Edition.* Washington, D.C.: American Association of State Highway and Transportation Officials, 1998.

- FHWA. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges.* Washington, D.C.: United States Department of Transportation, 1995.
  - http://www.fhwa.dot.gov/bridge/mtguide.pdf

- FHWA. *Bridge Inspector's Reference Manual.* Washington, D.C.: United States Department of Transportation, 2002.

- AASHTO. *Manual for Condition Evaluation of Bridges, 2nd Edition.* Washington, D.C.: American Association of State Highway and Transportation Officials, 2000.

- AASHTO. *CoRe Elements.* Washington, D.C.: American Association of State Highway and Transportation Officials, 2000.

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Section 1
Bridge Inspection Programs

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Topic 1.2 Responsibilities of the Bridge Inspector

1.2.1 Introduction

Bridge inspection has played, and will continue to play, an increasingly important role in providing a safe infrastructure for our nation. As our nation's bridges continue to age and deteriorate, an accurate and thorough assessment of each bridge's condition is critical in maintaining a safe, functional and reliable highway system.

This section presents the responsibilities of the bridge inspector. It also describes how the inspector can prepare for the inspection and some of the major inspection procedures.

1.2.2 Responsibilities of the Bridge Inspector and Engineer

There are five basic responsibilities of the bridge inspector and engineer:

- Maintain public safety and confidence
- Protect public investment
- Provide bridge inspection program support
- Provide accurate bridge records
- Fulfill legal responsibilities

1. Maintain Public Safety and Confidence

The primary responsibility of the bridge inspector is to maintain public safety and confidence. This is also a prime concern to everyone in the highway agency. The general public travels our highways and bridges without hesitation. However, when a bridge fails, the public's confidence in our bridge system is violated (see Figure 1.2.1). The design engineer's role in assuring bridge safety is:

- To incorporate safety factors.
- To provide cost-effective designs.

  Engineers provide a margin of safety to compensate for a lack of precise calculations, variations in the quality of material, erection loading conditions, and uncertain maintenance. This is particularly evident in older bridges, especially those designed prior to the use of computers. The bridge design engineer must be as confident as possible that the bridge will never fail under natural or man-made loads.

The inspector's role is:

- To provide thorough inspections identifying bridge conditions and defects,
- To prepare condition reports documenting these deficiencies and alerting supervisors or engineers of any findings which might impact the safety of the roadway user or the integrity of the structure.

Figure 1.2.1 Bridge Failure

2. Protect Public Investment

Another responsibility is to protect public investment in bridges. The inspector must be on guard for minor problems that can be corrected before they lead to costly major repairs. The inspector must also be able to recognize bridge elements that need repair in order to maintain bridge safety and avoid replacement costs.

As stated before, the funding available to rehabilitate and replace deficient bridges is not adequate to meet all of the needs. It is important that preservation activities be a part of the bridge program to extend the performance life of as many bridges as possible and minimize the need for costly repairs or replacement.

The inspector's role is to:

- Continually be on guard for minor problems that can become costly repairs.
- Recognize bridge components that need repair in order to maintain bridge safety and avoid the need for costly replacement.

The engineer's role is to:

- Continually upgrade design standards to promote longevity of bridge performance.

3. Provide Bridge Inspection Program Support

Subpart C of the National Bridge Inspection Standards (NBIS) of the *Code of Federal Regulations*, 23 Highways Part 650, mandates:

- Inspection procedures
- Frequency of inspections
- Qualifications of personnel
- Reporting
- Inventory

Bridge Inspection Programs are funded by public tax dollars. Therefore, the bridge inspector is financially responsible to the public.

The "Surface Transportation Act of 1978" established the funding mechanism for providing federal funds for bridge replacement. The Act also established criteria for bridge inspections and requirements for compliance with the NBIS.

The "Intermodal Surface Transportation Efficiency Act" (ISTEA) of 1991 and the Transportation Equity Act for the 21st Century (TEA-21) of 1998 establish funding mechanisms for tolled and free bridges for bridge maintenance, rehabilitation and replacement to adequately preserve the bridges and their safety to all users.

4. Provide Accurate Bridge Records

There are three major reasons why accurate bridge records are required:

a. To establish and maintain a structure history file.

For example, two bridge abutments are measured for tilt during several inspection cycles, and the results are as follows:

Year	Abutment A	Abutment B
2000	106 mm (4-3/16")	89 mm (3-1/2")
1998	106 mm (4-3/16")	57 mm (2-1/4")
1996	105 mm (4-1/8")	29 mm (1-1/8")
1994	102 mm (4")	25 mm (1")

Looking at year 2000 measurements only would indicate that Abutment A has a more severe problem. However, examining the changes each year, we see that the movement of A is slowing and may have stopped, while B is changing at a faster pace each inspection cycle. At the rate it is moving, B will probably surpass A by the next inspection.

b. To identify and assess bridge deficiencies and to identify and assess bridge repair requirements. An individual should be able to readily determine, from the records, what repairs are needed as well as a good estimate of quantities.

c. To identify and assess minor bridge deficiencies and to identify and assess bridge maintenance needs in a similar manner to the repair requirements.

To ensure accurate bridge records, proper record keeping needs to be maintained. A system should be developed to review bridge data and evaluate quality of bridge inspections.

5. Fulfill Legal Responsibilities

A bridge inspection report is a legal document. Descriptions must be specific, detailed, quantitative (where possible), and complete. Vague adjectives such as good, fair, poor, and general deterioration, without concise descriptions to back them up, should not be used. To say "the bridge is OK" is just not good enough.

Example of inspection descriptions:
Bad description: "Fair beams"
Good description: "Stringers in fair condition with light scaling on bottom flanges

of Beams B and D for their full length"
Bad description: "Deck in poor condition"
Good description: "Deck in poor condition with spalls covering 50% of the deck as indicated on field sketch, see Figure 42"

Any visual assessments should include phrases such as "no other apparent defects" or "no other defects observed."

Original inspection notes should not be altered without consultation with the inspector who wrote the notes.

A bridge inspection report implies that the inspection was performed in accordance with the National Bridge Inspection Standards, unless specifically stated otherwise in the report. Proper equipment, techniques, and personnel must be used. If the inspection is a special or interim inspection, this must be explained explicitly in the report.

1.2.3 Qualifications of Bridge Inspectors

The NBIS are very specific with regard to the qualifications of bridge inspectors. The *Code of Federal Regulations*, Title 23, Chapter 1, Section 650-307, (23 CFR 1.650.307), lists the qualifications of personnel for the National Bridge Inspection Standards. These are minimum standards; therefore, state or local highway agencies can implement higher requirements.

Inspection Program Manager

(a) The individual in charge of the organizational unit that has been delegated the responsibilities for bridge inspection, reporting, and inventory shall possess the following minimum qualifications:

 (1) Be a registered professional engineer; or

 (2) Be qualified for registration as a professional engineer under the laws of the State; or

 (3) Have a minimum of 10 years experience in bridge inspection assignments in a responsible capacity and have completed a comprehensive training course based on the "Bridge Inspector's Reference Manual," which has been developed by a joint Federal-State task force, and subsequent additions to the manual.

Inspection Team Leader

(b) An individual in charge of a bridge inspection team shall possess the following minimum qualifications:

(1) Have the qualifications specified in paragraph (a) of this section; or

(2) Have a minimum of 5 years experience in bridge inspection assignments in a responsible capacity and have completed a comprehensive training course based on the "Bridge Inspector's Reference Manual," which has been developed by a joint Federal-State task force.

(3) Current certification as a Level III or IV Bridge Safety Inspector under the National Society of Professional Engineers' program for the National Institute For Certification in Engineering Technologies (NICET) is an alternate acceptable means for establishing that a bridge inspection team leader is qualified.

Qualifications and responsibilities of inspection personnel can be found in Section 3.4 of the AASHTO Manual for Condition Evaluation of Bridges, 2nd Ed.

1.2.4 Consequence of Irresponsibility

The dictionary defines tort as "a wrongful act for which a civil action will lie except one involving a breach of contract."

In the event of negligence in carrying out the basic responsibilities described above, an individual, including department heads, engineers, and inspectors, is subject to personal liability. An inspector should strive to be as objective and complete as possible. Accidents that result in litigation are generally related, but not necessarily limited, to the following:

- Deficient safety features
- Failed members
- Failed substructure elements
- Failed joints or decks
- Potholes or other hazards to the traveling public
- Improper or deficient load posting

Anything said or written in the bridge file could be used in litigation cases held against you. In litigation involving a bridge, the inspection notes and reports may be used as evidence. A subjective report may have negative consequences for the highway agency involved in lawsuits involving bridges. The report will be scrutinized to determine if conditions are documented thoroughly and for the "proper" reasons. An inspector should, therefore, strive to be as objective and complete as possible. State if something could not be inspected.

Example of liabilities:

In a recent case, a consulting firm was found liable for negligent inspection practices. A tractor-trailer hit a large hole in a bridge deck, swerved, went through the guard rail, and fell 9.1 m (30 feet) to the ground. Ten years prior to the accident, the consulting firm had noted severe deterioration of the deck and had recommended tests to determine the need for replacement. Two years prior to the accident, their annual inspection report did not show the deterioration or recommend repairs. One year before the accident, inspectors from the consultant checked 345 bridges in five days, including the bridge on which the accident occurred. The court found that the consulting firm had been negligent in its inspection, and assessed the firm 75% of the ensuing settlement.

In another case, four cars drove into a hole 3.7 m (12 feet) deep and 9.1 m (30 feet) across during the night. Five people were killed and four were injured. The hole was the result of a collapse of a multi-plate arch. Six lawsuits were filed and, defendants included the county, the county engineer, the manufacturer, the supplier, and the consulting engineers who inspected the arch each year. The arch was built and backfilled, with mostly clay, by a county maintenance crew 16 years prior to the accident. Three years later, the county engineer found movement of 75 to 100 mm (3 to 4 inches) at one headwall. The manufacturer sent an inspector, who determined that the problem was backfill-related and recommended periodic measurements. These measurements were done once, but the arch was described as "in good condition" or "in good condition with housekeeping necessary" on subsequent inspections. Inspection reports documented a 150 mm (6 inch) gap between the steel plate and the headwall. A contractor examined the arch at the county engineer's request to provide a proposal for shoring. The county engineer discussed the proposal with the consulting engineers a month before the accident. Thirteen inspections in all were conducted on the structure. An engineering report accuses the county engineer of poor engineering practice.

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Table 4C Design Values for Mechanically Graded Dimension Lumber[1,2,3]

(Tabulated design values are for normal load duration and dry service conditions, unless specified otherwise. See NDS 2.3 for a comprehensive description of design value adjustment factors.)

USE WITH TABLE 4C ADJUSTMENT FACTORS

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)				Grading Rules Agency
		Bending F_b	Tension parallel to grain F_t	Compression parallel to grain F_c	Modulus of Elasticity E	
MACHINE STRESS RATED (MSR) LUMBER						
900f-1.0E		900	350	1050	1,000,000	WCLIB, WWPA
1200f-1.2E		1200	600	1400	1,200,000	NLGA, WCLIB, WWPA
1250f-1.4E		1250	800	1475	1,400,000	WCLIB
1350f-1.3E		1350	750	1600	1,300,000	NLGA, WCLIB, WWPA
1400f-1.2E		1400	800	1600	1,200,000	NLGA
1450f-1.3E		1450	800	1625	1,300,000	NLGA, WCLIB, WWPA
1500f-1.3E		1500	900	1650	1,300,000	WWPA
1500f-1.4E		1500	900	1650	1,400,000	NLGA, WCLIB, WWPA
1600f-1.4E		1600	950	1675	1,400,000	NLGA
1650f-1.3E		1650	1020	1700	1,300,000	NLGA, WWPA
1650f-1.5E		1650	1020	1700	1,500,000	NLGA, SPIB, WCLIB, WWPA
1650f-1.6E		1650	1175	1700	1,600,000	WCLIB, WWPA
1700f-1.6E		1700	1175	1725	1,600,000	WCLIB
1750f-2.0E		1750	1125	1725	2,000,000	WCLIB
1800f-1.5E		1800	1300	1750	1,500,000	NLGA, WWPA
1800f-1.6E		1800	1175	1750	1,600,000	NLGA, SPIB, WCLIB, WWPA
1950f-1.5E	2"& less in thickness	1950	1375	1800	1,500,000	SPIB, WWPA
1950f-1.7E		1950	1375	1800	1,700,000	NLGA, SPIB, WCLIB, WWPA
2000f-1.6E		2000	1300	1825	1,600,000	NLGA
2100f-1.8E	2"& wider	2100	1575	1875	1,800,000	NLGA, SPIB, WCLIB, WWPA
2250f-1.7E		2250	1750	1925	1,700,000	NLGA, WWPA
2250f-1.8E		2250	1750	1925	1,800,000	NLGA, WCLIB, WWPA
2250f-1.9E		2250	1750	1925	1,900,000	NLGA, SPIB, WCLIB, WWPA
2400f-1.8E		2400	1925	1975	1,800,000	NLGA, WWPA
2400f-2.0E		2400	1925	1975	2,000,000	NLGA, SPIB, WCLIB, WWPA
2500f-2.2E		2500	1750	2000	2,200,000	WCLIB
2550f-2.1E		2550	2050	2025	2,100,000	NLGA, SPIB, WCLIB, WWPA
2700f-2.0E		2700	1800	2100	2,000,000	WCLIB, WWPA
2700f-2.2E		2700	2150	2100	2,200,000	NLGA, SPIB, WCLIB, WWPA
2850f-2.3E		2850	2300	2150	2,300,000	NLGA, SPIB, WCLIB, WWPA
3000f-2.4E		3000	2400	2200	2,400,000	NLGA, SPIB
MACHINE EVALUATED LUMBER (MEL)						
M-5		900	500	1050	1,100,000	SPIB
M-6		1100	600	1300	1,000,000	SPIB
M-7		1200	650	1400	1,100,000	SPIB
M-8		1300	700	1500	1,300,000	SPIB
M-9		1400	800	1600	1,400,000	SPIB
M-10		1400	800	1600	1,200,000	NLGA, SPIB
M-11		1550	850	1675	1,500,000	NLGA, SPIB
M-12		1600	850	1675	1,600,000	NLGA, SPIB
M-13		1600	950	1675	1,400,000	NLGA, SPIB
M-14		1800	1000	1750	1,700,000	NLGA, SPIB
M-15		1800	1100	1750	1,500,000	NLGA, SPIB
M-16		1800	1300	1750	1,500,000	SPIB
M-17[4]	2"& less in thickness	1950	1300	2050	1,700,000	SPIB
M-18		2000	1200	1825	1,800,000	NLGA, SPIB
M-19	2"& wider	2000	1300	1825	1,600,000	NLGA, SPIB
M-20[4]		2000	1600	2100	1,900,000	SPIB
M-21		2300	1400	1950	1,900,000	NLGA, SPIB
M-22		2350	1500	1950	1,700,000	NLGA, SPIB
M-23		2400	1900	1975	1,800,000	NLGA, SPIB
M-24		2700	1800	2100	1,900,000	NLGA, SPIB
M-25		2750	2000	2100	2,200,000	NLGA, SPIB
M-26		2800	1800	2150	2,000,000	NLGA, SPIB
M-27[4]		3000	2000	2400	2,100,000	SPIB
M-28		2200	1600	1900	1,700,000	SPIB
M-29		1550	850	1650	1,700,000	SPIB

Table 4D Design Values for Visually Graded Timbers (5" x 5" and larger)

(Tabulated design values are for normal load duration and dry service conditions, unless specified otherwise. See NDS 2.3 for a comprehensive description of design value adjustment factors.)

USE WITH TABLE 4D ADJUSTMENT FACTORS

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)						Grading Rules Agency
		Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	
BALSAM FIR								
Select Structural	Beams and Stringers	1350	900	65	305	950	1,400,000	NELMA NSLB
No.1		1100	750	65	305	800	1,400,000	
No.2		725	350	65	305	500	1,100,000	
Select Structural	Posts and Timbers	1250	825	65	305	1000	1,400,000	
No.1		1000	675	65	305	875	1,400,000	
No.2		575	375	65	305	400	1,100,000	
BEECH-BIRCH-HICKORY								
Select Structural	Beams and Stringers	1650	975	90	715	975	1,500,000	NELMA
No.1		1400	700	90	715	825	1,500,000	
No.2		900	450	90	715	525	1,200,000	
Select Structural	Posts and Timbers	1550	1050	90	715	1050	1,500,000	
No.1		1250	850	90	715	900	1,500,000	
No.2		725	475	90	715	425	1,200,000	
COAST SITKA SPRUCE								
Select Structural	Beams and Stringers	1150	675	60	455	775	1,500,000	NLGA
No.1		950	475	60	455	650	1,500,000	
No.2		625	325	60	455	425	1,200,000	
Select Structural	Posts and Timbers	1100	725	60	455	825	1,500,000	
No.1		875	575	60	455	725	1,500,000	
No.2		525	350	60	455	500	1,200,000	
DOUGLAS FIR-LARCH								
Dense Select Structural	Beams and Stringers	1900	1100	85	730	1300	1,700,000	WCLIB
Select Structural		1600	950	85	625	1100	1,600,000	
Dense No.1		1550	775	85	730	1100	1,700,000	
No.1		1350	675	85	625	925	1,600,000	
No.2		875	425	85	625	600	1,300,000	
Dense Select Structural	Posts and Timbers	1750	1150	85	730	1350	1,700,000	
Select Structural		1500	1000	85	625	1150	1,600,000	
Dense No.1		1400	950	85	730	1200	1,700,000	
No.1		1200	825	85	625	1000	1,600,000	
No.2		750	475	85	625	700	1,300,000	
Dense Select Structural	Beams and Stringers	1850	1100	85	730	1300	1,700,000	WWPA
Select Structural		1600	950	85	625	1100	1,600,000	
Dense No.1		1550	775	85	730	1100	1,700,000	
No.1		1350	675	85	625	925	1,600,000	
Dense No.2		1000	500	85	730	700	1,400,000	
No.2		875	425	85	625	600	1,300,000	
Dense Select Structural	Posts and Timbers	1750	1150	85	730	1350	1,700,000	
Select Structural		1500	1000	85	625	1150	1,600,000	
Dense No.1		1400	950	85	730	1200	1,700,000	
No.1		1200	825	85	625	1000	1,600,000	
Dense No.2		800	550	85	730	550	1,400,000	
No.2		700	475	85	625	475	1,300,000	
DOUGLAS FIR-LARCH (NORTH)								
Select Structural	Beams and Stringers	1600	950	85	625	1100	1,600,000	NLGA
No.1		1300	675	85	625	925	1,600,000	
No.2		875	425	85	625	600	1,300,000	
Select Structural	Posts and Timbers	1500	1000	85	625	1150	1,600,000	
No.1		1200	825	85	625	1000	1,600,000	
No.2		725	475	85	625	700	1,300,000	
DOUGLAS FIR-SOUTH								
Select Structural	Beams and Stringers	1550	900	85	520	1000	1,200,000	WWPA
No.1		1300	625	85	520	850	1,200,000	
No.2		825	425	85	520	525	1,000,000	
Select Structural	Posts and Timbers	1400	950	85	520	1050	1,200,000	
No.1		1150	775	85	520	925	1,200,000	
No.2		650	400	85	520	425	1,000,000	

Table 5A Design Values for Structural Glued Laminated Softwood Timber

(Members stressed primarily in bending) [1,2,3,4,12] (Tabulated design values are for normal load duration and dry service conditions. See NDS 2.3 for a comprehensive description of design value adjustment factors.)

Use with Table 5A Adjustment Factors

Design values in pounds per square inch (psi)

		BENDING ABOUT X-X AXIS (Loaded Perpendicular to Wide Faces of Laminations)						BENDING ABOUT Y-Y AXIS (Loaded Parallel to Wide Faces of Laminations)					AXIALLY LOADED		
		Bending		Compression Perpendicular to Grain											
Combination Symbol[4]	Species Outer Lams/ Core Lams[5]	Tension Zone Stressed in Tension F_{bxx}	Compression Zone Stressed in Tension[6] F_{bxx}	Tension Face[9,10] $F_{c\perp xx}$	Compression Face[9,10] $F_{c\perp xx}$	Shear Parallel to Grain[10] F_{vxx}	Modulus of Elasticity E_{xx}	Bending F_{byy}	Compression Perpendicular to Grain (Side Faces) $F_{c\perp yy}$	Shear Parallel to Grain F_{vyy}	Shear Parallel to Grain (For Members With Multiple Piece Laminations Which are Not Edge Glued)[13] F_{vyy}	Modulus of Elasticity E_{yy}	Tension Parallel to Grain F_t	Compression Parallel to Grain F_c	Modulus of Elasticity E
VISUALLY GRADED WESTERN SPECIES															
16F-V1	DF/WW	1600	800	560[9,10]	560[10]	140	1,300,000	950	255	130[14]	65[14]	1,100,000	675	975	1,100,000
16F-V2	HF/HF	1600	800	500[10]	375[10]	155	1,400,000	1250	375	135	70	1,300,000	875	1300	1,300,000
16F-V3	DF/DF	1600	800	560[9,10]	560[10]	190	1,500,000	1450	560	165	85	1,500,000	950	1550	1,500,000
16F-V4[7]	DF/WW	1600	800	650	560[10]	90[10]	1,500,000	900	255	130[14]	65[14]	1,300,000	650	600	1,300,000
16F-V5[7]	DF/DF	1600	800	650	560[10]	90[10]	1,600,000	1000	470	135	70	1,500,000	750	875	1,500,000
16F-V6[8]	DF/DF	1600	1600	560[9,10]	560[10]	190	1,500,000	1450	560	165	85	1,400,000	950	1550	1,400,000
16F-V7[8]	HF/HF	1600	1600	375[10]	375[10]	155	1,400,000	1200	375	135	70	1,300,000	850	1350	1,300,000
20F-V1	DF/WW	2000	1000	650	560[10]	140	1,400,000	1000	255	130[14]	65[14]	1,200,000	750	1000	1,200,000
20F-V2	HF/HF	2000	1000	500[10]	375[10]	155	1,500,000	1200	375	135	70	1,400,000	950	1350	1,400,000
20F-V3	DF/DF	2000	1000	650	560[10]	190	1,600,000	1450	560	165	85	1,500,000	1000	1550	1,500,000
20F-V4	DF/DF	2000	1000	590[9,10]	560[10]	190	1,600,000	1450	560	165	85	1,600,000	1000	1550	1,600,000
20F-V5[7]	DF/WW	2000	1000	650	560[10]	90[10]	1,600,000	1000	255	135[14]	70[14]	1,300,000	750	725	1,300,000
20F-V7[8]	DF/DF	2000	2000	650	650	190	1,600,000	1450	560	165	85	1,600,000	1000	1600	1,600,000
20F-V8[8]	DF/DF	2000	2000	590[9,10]	590[9,10]	190	1,700,000	1450	560	165	85	1,600,000	1000	1600	1,600,000
20F-V9[8]	HF/HF	2000	2000	500[10]	500[10]	155	1,500,000	1400	375	135	70	1,400,000	975	1400	1,400,000
20F-V12	AC/AC	2000	1000	560	560	190	1,500,000	1200	470	165	80	1,400,000	900	1500	1,400,000
22F-V1	DF/WW	2200	1100	650	560[10]	140	1,600,000	1050	255	130[14]	65[14]	1,300,000	850	1100	1,300,000
22F-V3	DF/DF	2200	1100	650	560[10]	190	1,700,000	1450	560	165	85	1,600,000	1050	1500	1,600,000
22F-V8[8]	DF/DF	2200	2200	590[9,10]	590[9,10]	190	1,700,000	1450	560	165	85	1,600,000	1050	1650	1,600,000
22F-V10	DF/DFS	2200	1100	650	560[10]	190	1,600,000	1600	500	165	85	1,300,000	1000	1400	1,300,000
24F-V1	DF/WW	2400	1200	650	650	140	1,700,000	1250	255	135[14]	70[14]	1,400,000	1000	1300	1,400,000
24F-V2	HF/HF	2400	1200	500[10]	500[10]	155	1,500,000	1250	375	135	70	1,400,000	950	1300	1,400,000
24F-V4	DF/DF	2400	1200	650	650	190	1,800,000	1500	560	165	85	1,600,000	1150	1650	1,600,000
24F-V5	DF/HF	2400	1200	650	650	155	1,700,000	1350	375	140	70	1,500,000	1100	1450	1,500,000
24F-V8[8]	DF/DF	2400	2400	650	650	190	1,800,000	1450	560	165	85	1,600,000	1100	1650	1,600,000
24F-V10[8]	DF/HF	2400	2400	650	650	155	1,800,000	1400	375	140	70	1,600,000	1150	1600	1,600,000
24F-V11	DF/DFS	2400	1200	650	560[10]	190	1,700,000	1600	500	165	85	1,400,000	1150	1700	1,400,000

Section 2

Bridge Materials

Topic 2.1 Timber

2.1.1 Introduction

Approximately 7% of the bridges listed in the National Bridge Inventory (NBI) are classified as timber bridges. Another 7% of the total have a timber deck supported by a steel superstructure. Many of these bridges are very old, but the use of timber structures is gaining new popularity with the use of engineered wood products. (see Figure 2.1.1). To preserve and maintain them, it is important that the bridge inspector understand the basic characteristics of wood. Timber Bridges Design, Construction, Inspection and Maintenance August 1992 manual published by the United States Department of Agriculture, Forest Service is an excellent reference to supplement timber information in this manual. To order call (304) 285-1591. The National Wood in Transportation website is www.fs.fed.us/na/wit.

Figure 2.1.1 Glued-laminated Modern Timber Bridge

2.1.2

Basic Shapes Used in Bridge Construction

Depending on the required structural capacities and geometric constraints, wood can be cut into various shapes.

Round

Because sawmills were not created yet, most early timber bridge members were made out of solid round logs. Logs were generally used as beams, or stacked and used as abutments and foundations. In some parks, log bridges can still be seen. Round timber members have been used as piles driven into the ground or waterway bed. Logs have also been used as retaining devices for embankment material.

Rectangular

Once sawmill operations gained prominence, rectangular timber members became commonplace. Rectangular timber members were easier to connect together due to the flat sides and can be used for decking, superstructure beams, arches and truss elements, curbs or railings, and retaining devices (see Figure 2.1.2).

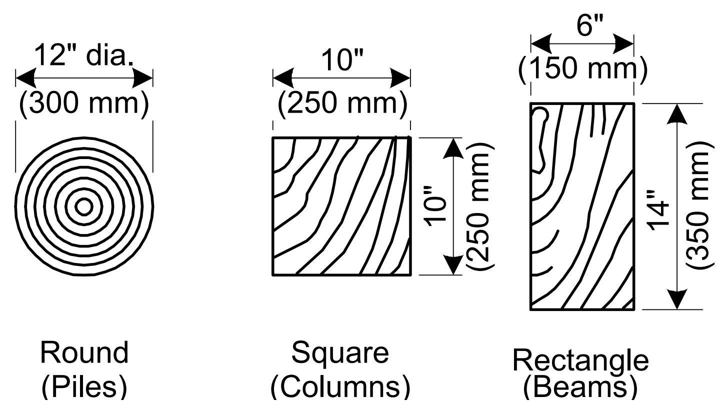

Figure 2.1.2 Timber Shapes

Built-up Shapes

Modern timber bridge members are fabricated from basic rectangular shapes to create built-up shapes, which perform at high capacities. A fundamental example of this is the slab-shaped beam. Two other common examples are T-shaped and box-shaped beams (see Figure 2.1.3). Using glue-laminate technology and stress timber design, these shapes enable modern timber bridges to carry current legal loads.

Refer to Section 6 for further information on timber superstructures and Topic 5.1 for timber decks.

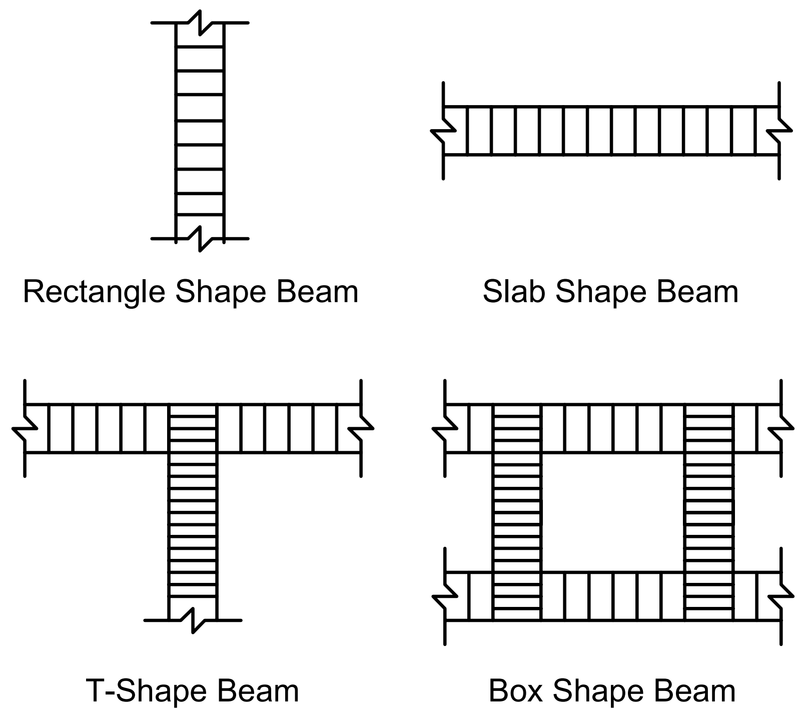

Figure 2.1.3 Built-up Timber Shapes

2.1.3 Properties of Timber

Because of its physical characteristics, wood is in many ways an excellent engineering material for use in bridges. Perhaps foremost is that it is a renewable resource. In addition, wood is:

- Strong, with a high strength to weight ratio
- Economical
- Aesthetically pleasing
- Readily available in many locations
- Easy to fabricate and construct
- Resistant to deicing agents
- Resistant to damage from freezing and thawing
- Able to sustain overloads for short periods of time (shock resistant)

However, wood also has some negative properties:

- Excessive creep under sustained loads
- Vulnerable to insect attack
- Vulnerable to fire

These characteristics stem from the unique physical and mechanical properties of wood, which vary with the species and grade of the timber.

Physical Properties

There are four basic physical properties that define timber behavior. These properties are classification, anatomy, growth features, and moisture content.

Timber Classification

Wood may be classified as hardwood or softwood. Hardwoods have broad leaves and lose their leaves at the end of each growing season. Softwoods, or conifers, have needle-like or scale-like leaves and are evergreens. The terms "hardwood" and "softwood" are misleading because they do not necessarily indicate the hardness or softness of the wood. Some hardwoods are softer than certain softwoods and vice versa.

Timber Anatomy

Wood is a non-homogeneous material. Wood, although an extremely complex organic material, has dominant and fundamental patterns to its cell structure. Some of the physical properties of this cell structure include (see Figure 2.1.4):

- Hollow cell composition - cell walls consist of cellulose and lignin, and are formed in an oval or rectangular shape which accounts for the high strength-to-weight ratio of wood; wood with thick cell walls is dense and strong; lignin bonds the cells together
- Growth rings - revealed in the cross section of a tree; they are distinct annual rings of wood, denser toward the end of each session, sometimes darker in color in that part of each ring (as in Douglas fir and southern pine), sometimes with little color difference (spruces and true firs); depends on species
- Sapwood - the active, outer part of the tree that conducts sap and stores food throughout the tree; is generally permeable and easier to treat with preservatives; sapwood is of lighter color than heartwood
- Heartwood - the inactive, inner part of the tree which serves to support the tree; may be resistant to decay due to toxic materials deposited in the heartwood cells; usually of darker color than sapwood
- Wood rays - groups of cells, running from the center of the tree horizontally to the bark, which are responsible for cross grain strength
- Grain - the wood fibers oriented along the long axis of logs and timbers; the direction of greatest strength

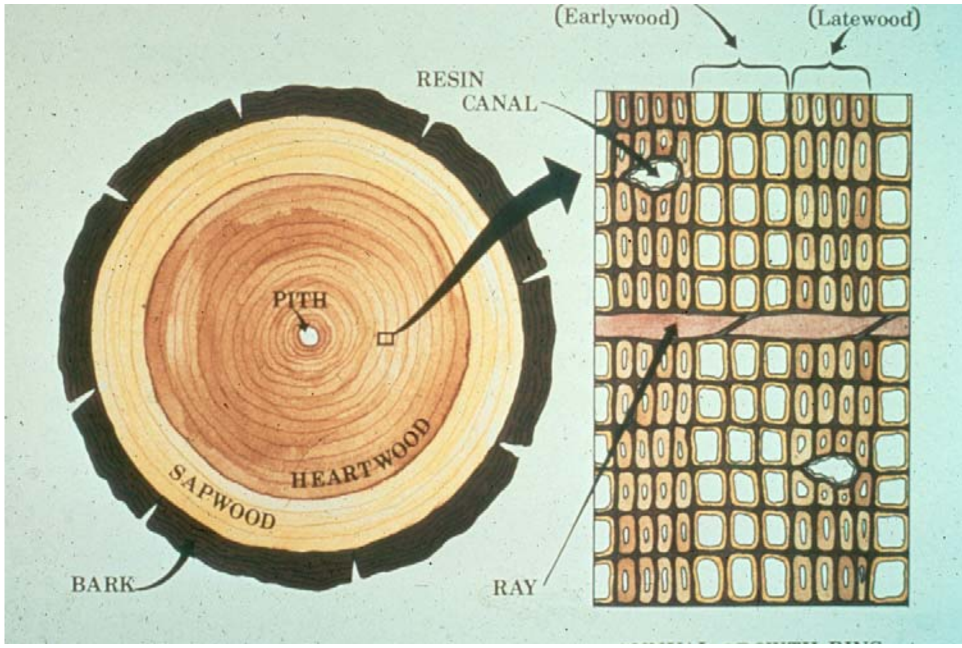

Figure 2.1.4 Anatomy of Timber

Growth Features

A variety of growth features adversely affect the strength of wood. Some of these features include:

- Knots and knot holes - due to intergrown limbs and associated grain deviation
- Sloping grain - caused by the normal taper of a tree or by sawing in a direction other than parallel to the grain
- Splits, checks, and shakes - separation of the cells along the grain, primarily due to rapid or uneven drying and differential shrinkage in the radial and tangential directions during seasoning; checks and splits occur across the growth rings; a shake is a type of check which occurs between the growth rings, peculiar to a few species
- Reaction wood - a type of abnormal wood that is formed in leaning trees; the pith is off center; the wood is gelatinous and displays cross grain shrinkage checks when seasoned

Moisture Content

Moisture content affects wood. It causes dimensional instability and fluctuations of weight and affects the strength and decay resistance of wood. It is most desirable for wood to have the least moisture content as is possible. This is done naturally over time (seasoning) or using kiln drying.

Mechanical Properties

In addition to the physical properties of timber, there are also four important mechanical properties which govern the use of timber in structures.

Orthotropic Behavior

Wood is considered a non-homogeneous and an orthotropic material. It is non-homogeneous because of the random occurrences of knots, splits, checks, and the variance in cell size and shape. It is orthotropic because wood has mechanical properties that are unique to its three principal axes of anatomical symmetry (longitudinal, radial, and tangential). This orthotropic behavior is due to the orientation of the cell fibers in wood (see Figure 2.1.5).

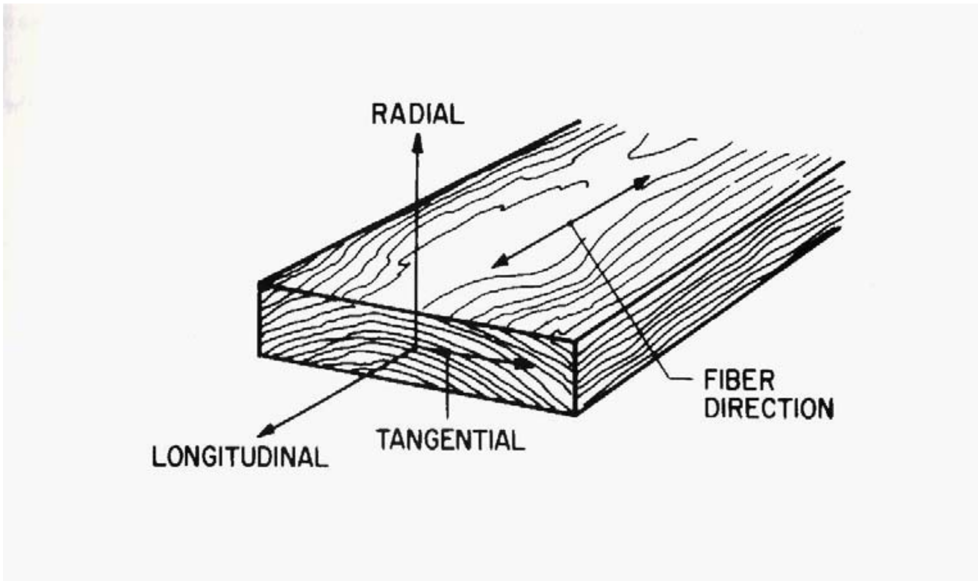

Figure 2.1.5 Three Principal Axes of Wood

As a result of its orthotropy, wood has three distinct sets of strength properties. Because timber members are longitudinal sections of wood, strength properties are commonly defined for the longitudinal axis. However, an exception is bearing strength perpendicular to the grain. American Society for Testing and Materials (ASTM) standards are issued which present strength properties for various types of wood.

Fatigue Characteristics

Because wood is a fibrous material, it tends to be less sensitive than steel or iron to repeated loads. Therefore, it is somewhat fatigue resistant. The presence of knots and sloping grain reduces the strength of wood considerably more than does fatigue; therefore, fatigue is generally not a limiting factor in timber design.

Impact Resistance

Wood is able to sustain short-term loads of about twice the level it can bear on a permanent basis, provided the cumulative duration of such loads is limited.

Creep Characteristics

Creep occurs when a load is maintained on wood. That is, the initial deflection of the member increases with time. Green timbers may sag appreciably, if allowed to season under load. Initial deflection of unseasoned wood under permanent loading can be expected to double with the passage of time. Therefore, to accommodate creep, twice the initial elastic deformation is often assumed for design. Partially seasoned material may also creep to some extent. However, thoroughly seasoned wood members will exhibit little permanent increase in deflection with time.

2.1.4 Timber Grading

The most widely used species of wood for bridge construction are Douglas fir and southern pines. The southern pines include several species graded and marketed under identical grading rules. Other species, such as western hemlock and eastern spruce, are suitable for bridge construction if appropriate allowable stresses are used. Some hardwoods are also used for bridge construction.

Timber is given a grading so that the following can be established:

- Modulus of elasticity
- Tensile stress parallel to grain
- Compressive stress parallel to grain
- Compressive stress perpendicular to grain
- Shear stress parallel to grain (horizontal shear)
- Bending stress

Timber used for outdoor applications needs to be designed for wet service condition. Refer to *Timber Bridges: Design, Construction, Inspection, and Maintenance*, Forest Service, United States Department of Agriculture.

The ultimate strength properties of wood in the tables at the beginning of this topic are for air-dried wood, which is clear, straight grained, and free of strength-reducing defects. Reduction factors need to be applied to these values based on use.

Preservative treatment for decay resistance does not alter the allowable stresses for design, provided any moisture associated with the treatment process is removed.

Unlike steel, the elastic modulus of wood varies with the grades and species.

Sawn Lumber

The grading of sawn timber is accomplished by either a visual grading or a mechanical stress grading (MSR). Refer to the tables at the beginning of this topic.

Visual Grading

This type of grading is the most common and is performed by a certified lumber grader. The lumber grader inspects each sawn and surfaced piece of lumber. The individual pieces of lumber must meet particular grade description requirements in order to be classified at a certain grade. If the requirements are not met, the piece of sawn and surfaced lumber is compared to lower grade description requirements until the piece of lumber fits into the appropriate grade. Mechanical properties are predetermined for each grade. Therefore, once the piece of lumber has been graded, the mechanical properties are known.

Mechanical Stress Grading

Mechanical stress grading or mechanical stress rating (MSR) grades lumber by the relationship between the modulus of elasticity and the bending strength of lumber. The machine measures the bending strength and then assigns an elastic modulus. The grading mainly depends on the elastic modulus but can be changed by visual observance of edge knots, checks, shakes, splits, and warps. Mechanical stress grading has a different set of grading symbols than visual grading.

Glued-Laminated Lumber

Glued-laminated lumber or glulam grades use a combination symbol that describes the combination of lamination grades. Bending and axial are the two types of combination grades. A glulam member will be graded under the bending combination if it is designed for use as a flexure member (see the tables at the beginning of this topic), or it will be graded under the axial combination if it is designed for use as an axial loaded member.

2.1.5 Types and Causes of Timber Deterioration

Although wood is an excellent material for use in bridges, untreated wood is vulnerable to damage from fungi, parasites, and other sources. The untreated inner cores of treated timbers and poles are vulnerable to these predators if they can gain access through the outer treated shell. The degree of vulnerability varies with the species and grade of the timber. Bridge inspectors must be able to recognize the signs of the various types of damage and be able to evaluate their effect on the structure.

Natural Defects

Defects that form from abnormal growth or from the lumber drying process include (see Figure 2.1.6):

- Checks - separations of the wood fibers, normally occurring across or through the annual growth rings, and generally parallel to the grain direction
- Splits - similar to checks except the separations of the wood fibers extend completely through the piece of wood; a split is also known as a through check
- Shakes - separations along the grain which occur between the annual growth rings

These three defects provide openings for decay to begin and in some cases indicate reduced strength in the member when the defect is in an advanced state.

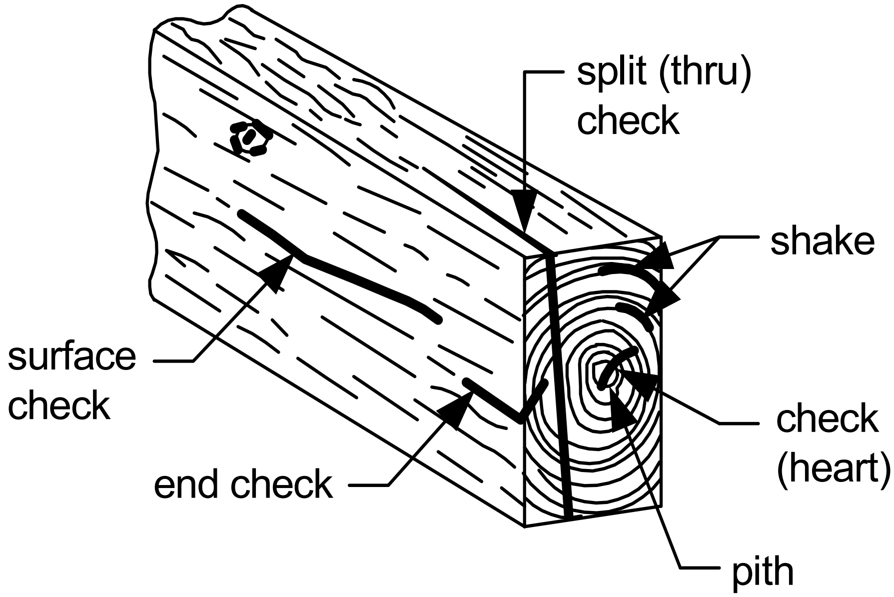

Figure 2.1.6 Natural Timber Defects

Fungi

Decay is the primary cause of timber bridge replacement. Decay is the process of living fungi, which are plants feeding on the cell walls of wood (see Figure 2.1.7). The initial process is started by the deposition of spores or microscopic seeds. Fruiting bodies (e.g., mushrooms and conks) produce these spores by the billions. The spores are distributed by wind, water, or insects.

Figure 2.1.7 Decay of Wood by Fungi

Spores that survive and experience favorable growth conditions can penetrate timber bridge members in a few weeks. Favorable conditions for fungi to grow can only occur when these four requirements exist:

- Oxygen - Sufficient oxygen must be available for the fungi to breathe. A minimal amount of free oxygen can sustain them in a dormant state, but at least 20 percent of the volume of wood must be occupied by air for fungi to become active. The air we breathe contains about 21 percent oxygen. Absence of oxygen in bridge members would only occur in piling or bents placed below the permanent low water elevation or water table, or buried in the ground.

- Temperature - A favorable temperature range must be available for the growth of fungi to occur. Below 0ºC (32ºF), the fungi become dormant but resumes its growth as the temperature rises above freezing to the 24ºC to 29ºC (75ºF to 85ºF) range, where growth is at its maximum. Above 32ºC (90ºF), growth tapers off rapidly, and temperatures in excess of 49ºC (120ºF) become lethal to the fungi. These killing temperatures could only occur in bridge members during kiln drying or preservative treating.

- Food - An adequate food supply must be available for the fungus to feed on. As the entire bridge serves as the food supply, the only prevention is to poison the wood supply with preservatives.

- Moisture - The fourth and probably the most controlling requirement is an adequate supply of moisture. The term "dry-rot" is misleading because dry wood will not rot. Wood must have a moisture content of 20 percent or greater for the growth of fungi to become active. Rain or snow is the main source of wood wetting. Secondary sources are condensation, ground water, and stream water. Exposed surfaces allow moisture to evaporate harmlessly. However seasoning checks, interfaces between timber members, and fastener holes are ideal for localized moisture accumulation which allow fungi to grow.

Although there are numerous types and species of fungi, only a few cause decay in timber bridge members. Some fungi types that do not cause damage include:

- Molds - cottony or powdery circular growths varying from white or light colors to black; molds themselves do not cause decay but their presence is an indication that conditions favorable to the growth of fungi exist (see Figure 2.1.8)
- Stains - specks, spots, streaks, or patches, varying in color, which penetrate the sap wood; sapstain is harmless to wood; it is usually a surface phenomenon and, like molds, implies conditions where harmful fungi can flourish
- Soft rot - attacks the wood, making it soft and spongy; only the surface wood is affected, and thus it does not significantly weaken the member; occurs mostly in wood of high water content and high nitrogen content

Some fungi types that weaken the wood include:

- Brown rot - degrades the cellulose and hemi-cellulose leaving the lignin as a framework which makes the wood dark brown and crumbly (see Figure 2.1.9)
- White rot - feeds upon the cellulose, hemi-cellulose, and the lignin and makes the wood white and stringy (see Figure 2.1.9)

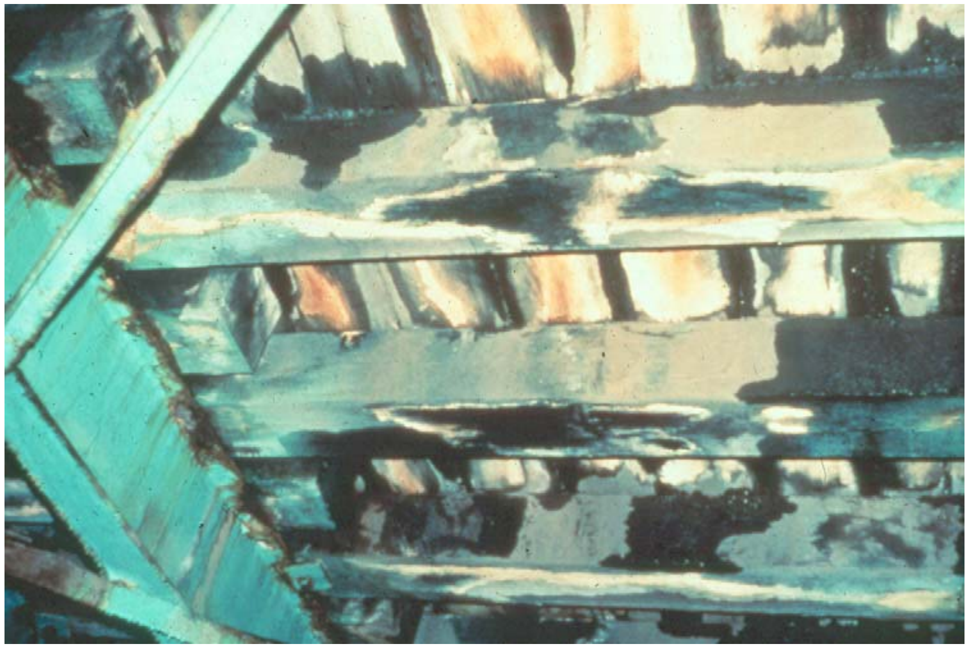

Figure 2.1.8 Mold and Stain on Underside of Timber Bridge

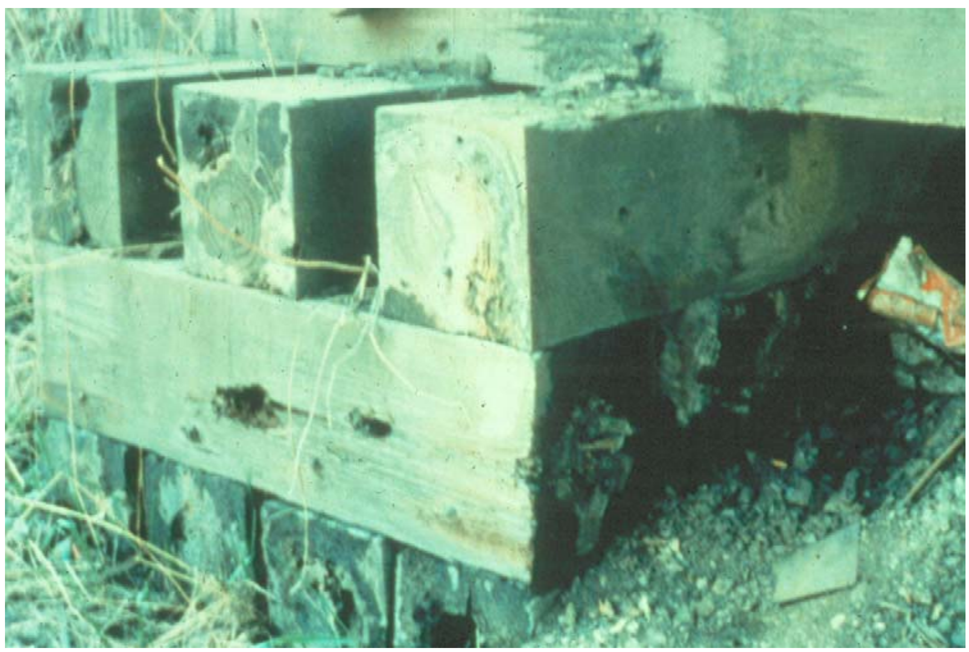

Figure 2.1.9 Brown and White Rot

Brown and white rots are responsible for structural damage to wood.

The natural decay resistance of wood exposed under conditions favorable for decay is distinctly variable, and it can be an important factor in the service life of wood bridges.

The heartwood of many tree species possesses a considerable degree of natural decay resistance, while the sapwood of all commercial species is vulnerable to decay.

Each year, when an inner layer or ring of sapwood dies and becomes heartwood, fungi-toxic compounds are deposited. These compounds provide natural decay resistance and are not present in living sapwood.

Most existing wood bridges in this country have been constructed from either Douglas fir or southern pine. Older bridges may contain such additional species as larch, various pines, and red oak. The above named species are classified as moderately decay resistant. Western red cedar and white oak are considered very decay resistant.

In the last 25 years, wood bridge materials have been obtained increasingly from smaller trees in young-growth timber stands. As a result, recent supplies of lumber and timbers have contained increased percentages of decay-susceptible sapwood.

Insects

Insects tunnel in and hollow out the insides of timber members for food and shelter. Some common types of insects include:

- Termites
- Carpenter ants
- Powder-post beetles or lyctus beetles
- Caddisflies

Termites

Termites are pale-colored, soft-bodied insects that feed on wood (see Figure 2.1.10). All damage is inside the surface of the wood; hence, it is not visible. The only visible signs of infestation are white mud shelter tubes or runways extending up from the earth to the wood and on the sides of masonry substructures. Termite attack of bridge members, however, is rare or nonexistent in bridges throughout most of the country due to the constant vibration caused by traffic travelling over timber bridges.

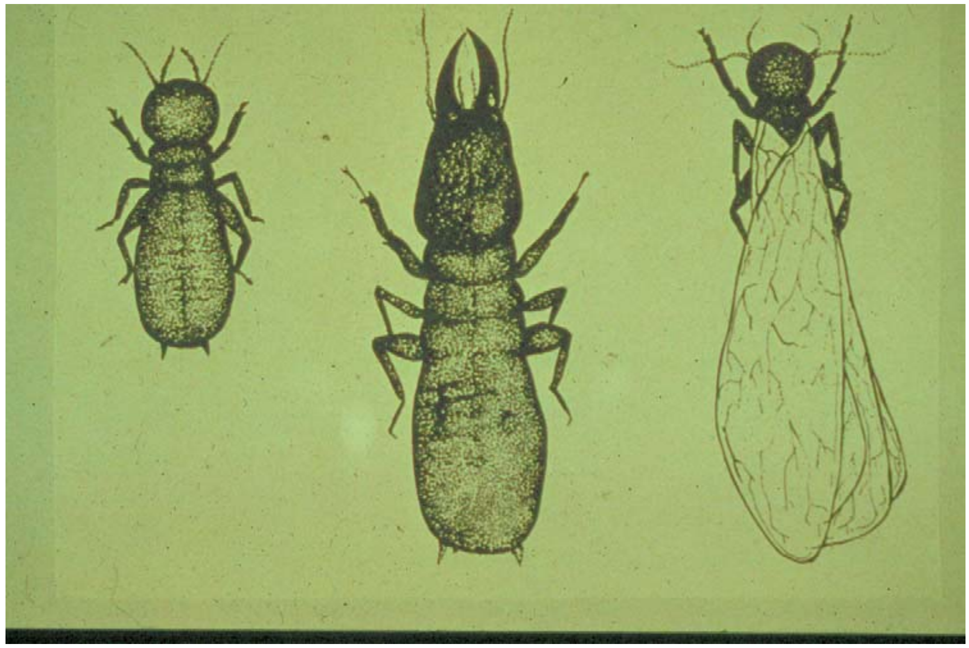

Figure 2.1.10 Termites

Carpenter Ants

Carpenter ants are large, black ants up to 3/4 inches (19 mm) long that gnaw galleries in soft or decayed wood (see Figure 2.1.11). The ants may be seen in the vicinity of the infested wood, but the accumulation of sawdust on the ground at the base of the timber is also an indicator of their presence. The ants do not use the wood for food but build their galleries in the moist and soft or partially decayed wood.

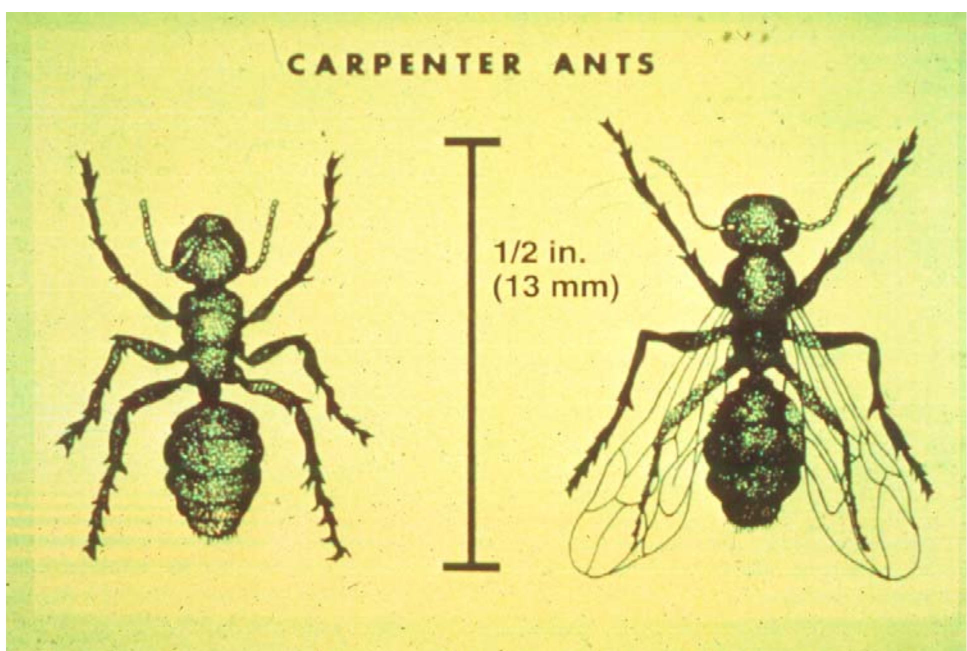

Figure 2.1.11 Carpenter Ants

Powder-post Beetles or Lyctus Beetles

Powder-post beetle larvae also hollow out the insides of timber members and leave the outer surface pocked with small holes. Often a powdery dust is dislodged from the holes. The inside may be completely excavated as the larvae of these beetles bore through the wood for food and shelter.

Caddisflies

The caddisfly is another insect that can damage timber piles. It is generally found in fresh water but can also be found in brackish water. Bacterial and fungal decay make the timber attractive to the caddisfly.

The caddisfly is an aquatic insect that is closely related to the moth and butterfly (see Figure 2.1.12). During the larva and pupa stage of their life cycle, they can dig small holes in the timber for protection. The larvae do not feed on the timber, but rather use it as a foundation for their silken shelters. This explains why caddisfly larvae have been known to exist on creosote treated timber.

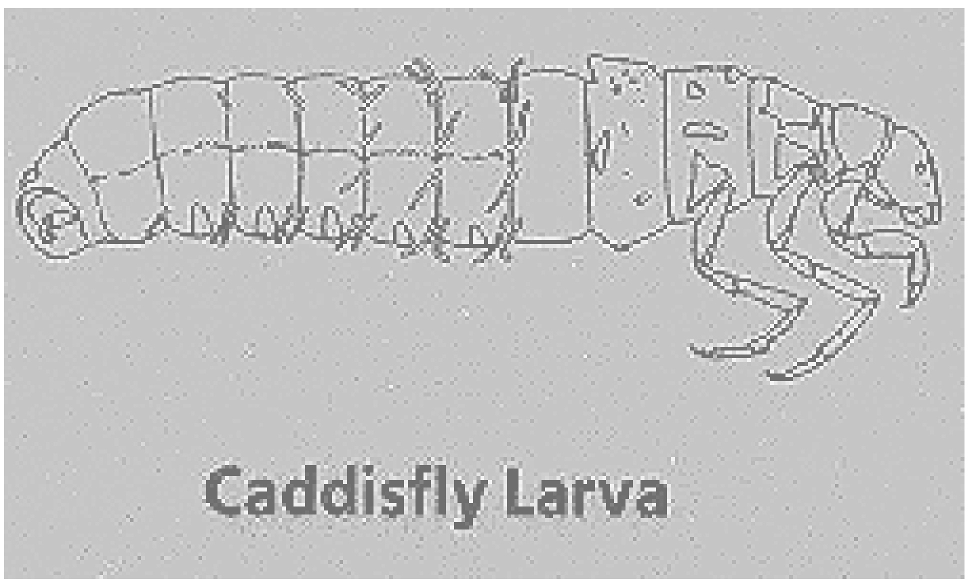

Figure 2.1.12 Caddisfly Larva

Marine Borers

Marine borers are found in sea water and brackish water only and cause severe damage to timber members in the area between high and low water, although damage may extend to the mud line (see Figure 2.1.13). They can be very destructive to wood and have been known to consume piles and framing in just a few months.

One type of marine borer is the mollusk borer, or shipworm (see Figure 2.1.14). The shipworm is one of the most serious enemies of marine timber installations. The most common species of shipworm is the teredo. This shipworm enters the timber in an early stage of life and remains there for the rest of its life. Teredos are gray and slimy and can typically reach a length of 380 mm (15 inches) and a diameter of 10 mm (3/8 inch). Some species of shipworm have been known to grow to a length of 1.8 m (6 feet) and up to 25 mm (1 inch) in diameter. The teredo maintains a small opening in the surface of the wood to obtain nourishment from the sea water.

Figure 2.1.13 Marine Borer Damage to Wood Piling

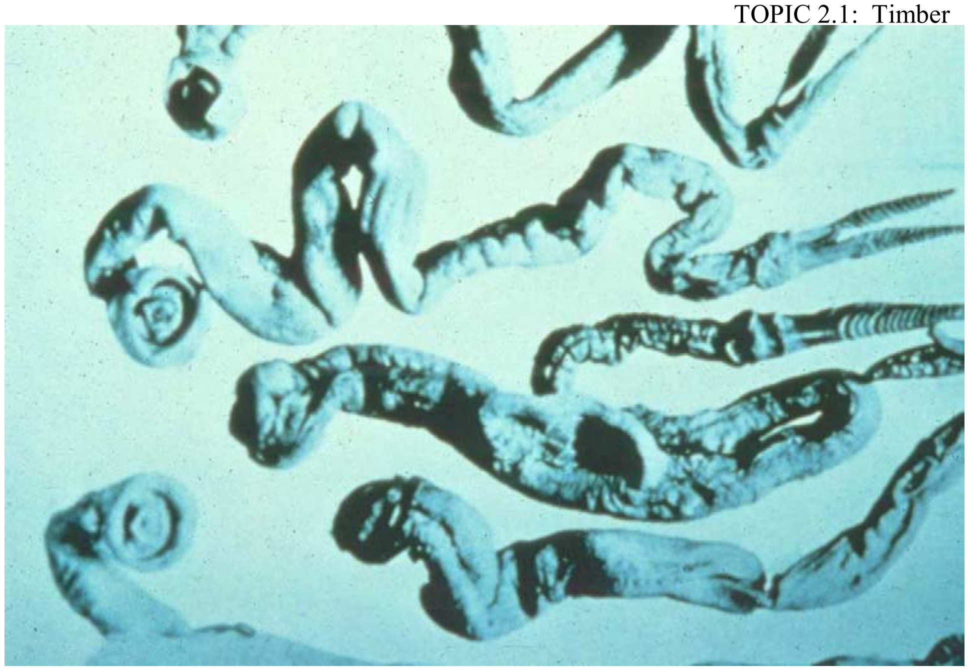

Figure 2.1.14 Shipworms (Mollusks)

Another type of marine borer is the crustacean borer. The most commonly encountered crustacean borer is the limnoria or wood louse (see Figure 2.1.15). It bores into the surface of the wood to a shallow depth. Wave action or floating debris breaks down the thin shell of timber outside the borers' burrows, causing the limnoria to burrow deeper. The continuous burrowing results in a progressive deterioration of the timber pile cross section, which will be noticeable by an hourglass shape developed between the tide levels. These borers are about 3 to 6 mm (1/8 to 1/4 inches) long and 2 to 3 mm (1/16 to 1/8 inches) wide.

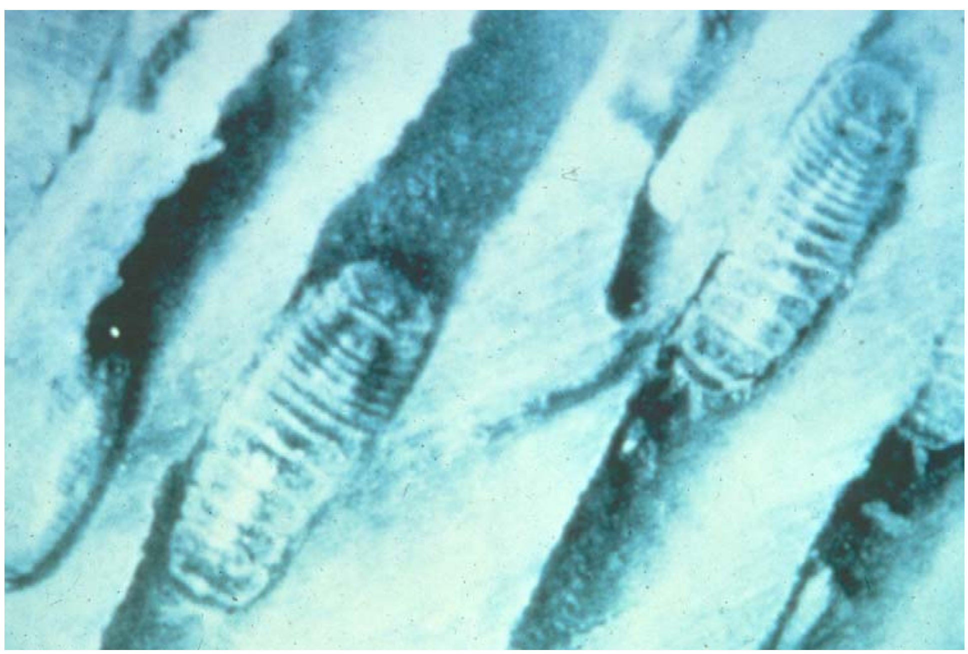

Figure 2.1.15 Limnoria Burrowing in Wood

Chemical Attack

Most petroleum based products and chemicals do not cause structural degradation to wood. However, animal waste can cause some damage, and strong alkalis will destroy wood fairly rapidly. Highway bridges are seldom exposed to these substances. Timber structures normally do not come in contact with damaging chemicals unless an accidental spill occurs.

Acids

Wood resists the effects of certain acids better than many materials and is often used for acid storage tanks. However, strong acids that have oxidizing properties, such as sulphuric and sulphureous acid, are able to slowly remove a timber structure's fiber by attacking the cellulose and hemi-cellulose. Acid damaged wood has weight and strength losses and looks as if it has been burned by fire.

Bases or Alkalis

Strong bases or alkalis attack and weaken the hemi-cellulose and lignin in the timber structure. Attack by strong bases leaves the wood a bleached white color. Mild alkalis do little harm to wood.

Other Types and Sources of Deterioration

Delaminations

Delaminations occur in glued-laminated members when the layers separate due to failure within the adhesive or at the bond between the adhesive and the laminate. They provide openings for decay to begin and may cause a reduction in strength (see Figure 2.1.16).

Loose connections

Loose connections may be due to shrinkage of the wood, crushing of the wood around the fastener, or from repetitive impact loading (working) of the connection. Loose connections can reduce the bridge's load-carrying capacity (see Figure 2.1.17).

Surface depressions

Surface depressions indicate internal collapse, which could be caused by decay.

Fire

Fire consumes wood at a rate of about 0.05 inches (1 mm) per minute during the first 30 minutes of exposure, and 0.021 inches (0.5 mm) per minute thereafter (see Figure 2.1.18). Large timbers build a protective coating of char (carbon) after the first 30 minutes of exposure. Small size timbers do not have enough volume to do this before they are, for all practical purposes, consumed. Preservative treatments are available to retard fire damage.

Figure 2.1.16 Delamination in a Laminated Timber Member

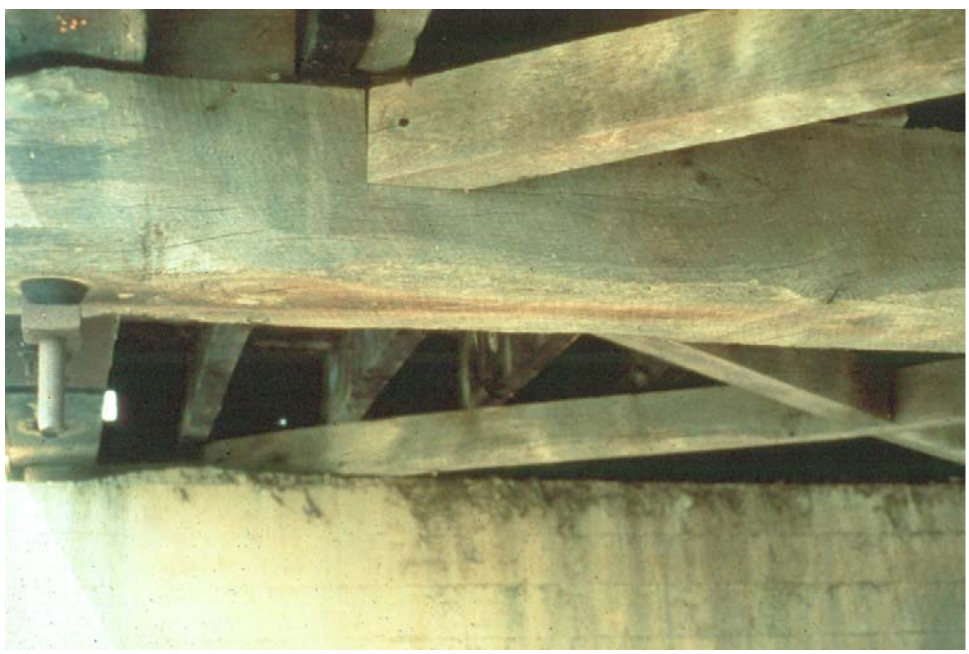

Figure 2.1.17 Hanger Connection on a Timber Floorbeam

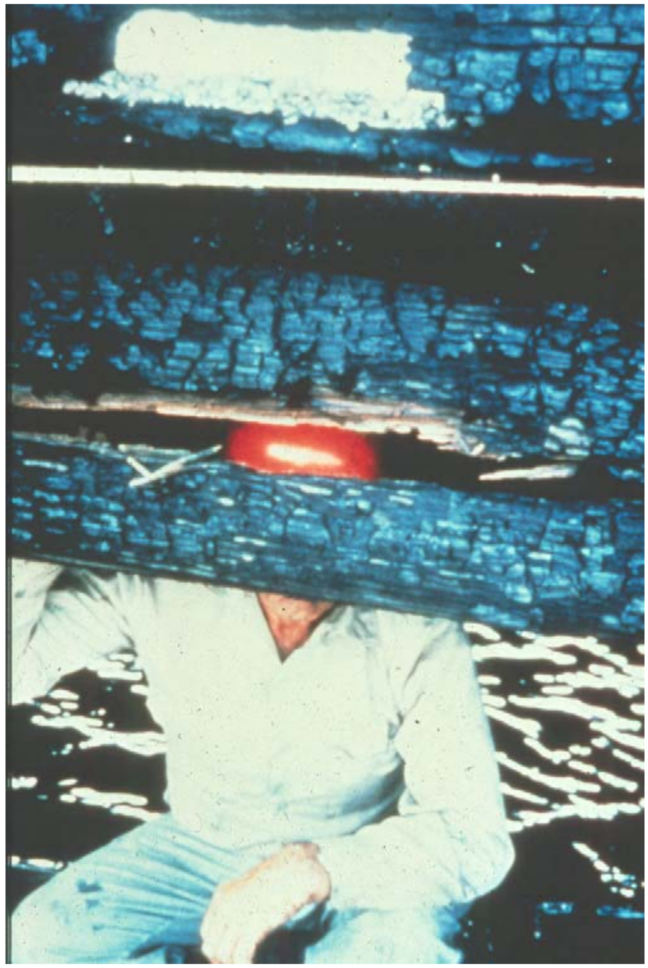

Figure 2.1.18 Fire Damaged Timber Member

Impact or Collisions

Severe damage can occur to truss members, railings, and columns when an errant vehicle strikes them (see Figure 2.1.19).

Figure 2.1.19 Impact/Collision Damage to a Timber Member

Abrasion or Mechanical Wear

Vehicular traffic is the main source of abrasion on timber decks (see Figure 2.1.20). Abrasion also occurs on timber piles that are subjected to tidal flows. Mechanical wear of timber members sometimes occurs due to movement of the fasteners against their holes when connections become loose.

Figure 2.1.20 Abrasion Damage on a Timber Deck

Overstress

Each timber member has a certain ultimate load capacity. If this load capacity is exceeded, the member will fail (see Figures 2.1.21 and 2.1.22).

Figure 2.1.21 Horizontal Shear Failure in Timber Member

Figure 2.1.22 Failed Timber Floor Beam

Weathering or Warping

Weathering is the affect of light, water, and heat. Weathering can change the equilibrium moisture content in the wood in a non-uniform fashion, thereby resulting in changes in the strength and dimensions of the wood. Uneven reduction in moisture content causes localized shrinkage, which can lead to warping, checking, splitting, or loosening of connectors (see Figure 2.1.23).

Figure 2.1.23 Weathering on Timber Deck

Protective Coating Failure

The following paint failures are common on wood:

- Cracking and peeling extend with the grain of the wood. They are caused by different shrink and swell rates of expansion and contraction between springwood (the lighter colored, wider spaces between the "rings" which grow in springtime) and denser summerwood (the darker, narrower rings which grow in hotter, drier summer).
- Decay fungi penetrate through cracks in the paint to cause wood to decay.
- Blistering is caused by paint applied over an improperly cleaned surface. Water, oil, or grease typically are responsible for blistering.
- Chalking is a degradation of the paint, usually by the ultraviolet rays of sunlight, leaving a powdery residue.
- Erosion is general thinning of the paint due to chalking, weathering, or abrasion.
- Mold fungi and stain fungi grow on the surface of paint, usually in warm, humid, shaded areas with low air flow. They appear as small green or black spots.

2.1.6 Protective Systems

Protective systems are a necessity when using timber for bridge construction. Proper preparation of the timber surface is required for the protective system to penetrate the wood surface and perform adequately. Untreated timber generally has a unit weight of about 640 to 800 kilograms per cubic meter (40 to 50 pounds per cubic foot (pcf)).

Types and Characteristics of Wood Protectants

Water Repellents

Water repellents prevent water absorption and maintain low moisture content in wood. This helps to prevent decay by molds and to slow the weathering process. Laminated wood (plywood) is particularly susceptible to moisture variations, which cause stress between plies due to swelling and shrinkage.

Preservatives

Wood preservatives prevent biological deterioration that can penetrate deep into timber. To be effective, the preservatives have to be applied to wood by vacuum-pressure treatment. This is done by placing the timber to be treated in a sealed chamber up to 2.4 m (8 feet) in diameter and 43 m (140 feet) long. The chamber is evacuated, drawing the air from the wood pores and cells. The treatment chemical is then fed into the chamber and pressure up to 1380 kPa (200 psi) is applied, forcing the chemical into the wood (see Figure 2.1.24). Preservatives are the best means to prevent decay but do not prevent weathering. A paint or water repellent coating is required for this. Treated timber generally has a unit weight of about 800 kilograms per cubic meter (50 pounds per cubic foot (pcf)).

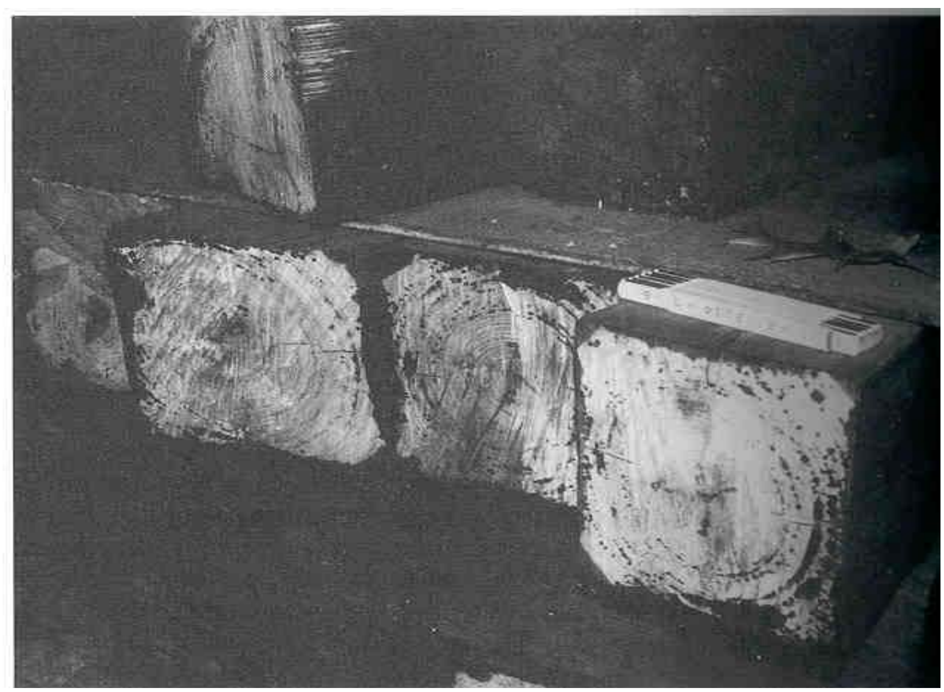

Figure 2.1.24 Bridge Timber Member Showing Penetration Depth of Preservative Treatment

Coal tar-creosote is a dark, oily protectant used in structural timber such as pilings and beams. Coal tar-creosote treated timber has a dark, oily appearance (see Figure 2.1.25). Unless it has weathered for several years, it cannot be painted, since paint adheres poorly to the oily surface, and the oils bleed through paint.

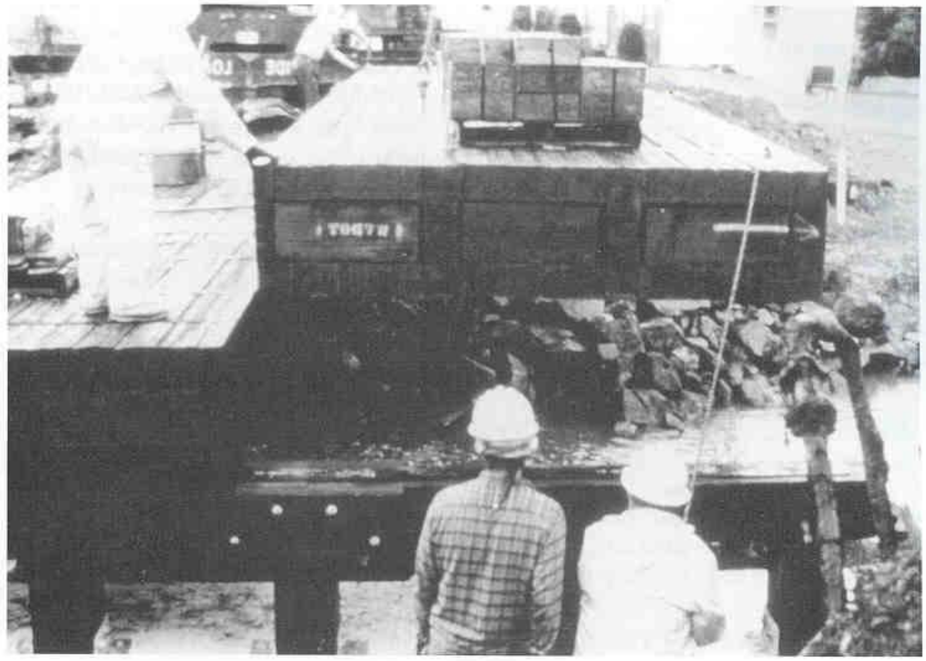

Figure 2.1.25 Coal-Tar Creosote Treated Timber Beams (Source: Barry Dickson, West Virginia University)

Pentachlorophenol (in a light oil solvent) is an organic solvent solution used as a decay inhibitor. It also leaves an oily surface, like creosote, but can be painted after all of the solvent has evaporated, usually in one or two years of normal service.

Chromated copper arsenate (CCA) is the most common waterborne salt decay inhibitor and is also applied by vacuum-pressure treatment. Timber treated with CCA has a green appearance. It is the only pressure-applied preservative that readily accepts painting. CCA also provides limited protection against the ultraviolet rays in sunlight.

Pole-fuming is used to kill decay fungi in timber pilings which are already in-service. The treatment chemical, injected through bore holes drilled into the piling, spreads along wood fibers for up to 2.7 m (9 feet) from the injection site. It stops existing decay and prevents further decay for up to nine years.

Fire Retardants

Fire retardants will not indefinitely prevent wood from burning but will retard the spread of fire and prolong the time to ignite wood. The two main classes of fire retardants are pressure impregnated fire retardant salts and intumescent coatings (paints). The intumescent paints expand upon intense heat exposure, forming a thick, puffy, charred coating which insulates the wood from the intense heat. Application of fire retardants may change some wood properties of glued-laminated timber.

Paint

Wood must be sufficiently dry to permit painting. A few months of seasoning will satisfactorily dry new wood. The wood surface must be free of dirt and debris prior to painting. Old, poorly adherent paint must be removed and the edges of intact paint feathered for a smooth finish. Mildew shows up as green or black spots on bare wood or paint. It is a fungus which typically grows in warm, humid, shaded areas with low air movement. Mildew must be removed with a solution of sodium hypochlorite (bleach) and water.

There are several common methods to prepare wood for painting:

- Hand tool cleaning is the simplest but slowest method. Sandpaper, scrapers, and wire brushes are used to clean small areas.
- Power tool cleaning utilizes powerized versions of the hand tools. They are faster than hand tools, but care must be exercised not to damage the wood substrate.
- Heat application with an electric heat gun softens old paint for easier removal to bare wood.
- Solvent-based and caustic chemical paint removers can efficiently clean large areas quickly. Some of the chemicals may, however, present serious fire or exposure hazards. Extreme caution must be exercised when working around chemical paint removers.
- Open nozzle abrasive blast cleaning and water blast cleaning remove old paint and foreign material, leaving bare wood. However, they can easily damage wood unless used carefully.

Paint protects wood from both moisture and weathering. By precluding moisture from wood, paint prevents decay. However, paint applied over unseasoned wood seals in moisture, accelerating, rather than retarding, decay. Oil-based paint and latex paint are both commonly used on wood bridges.

Oil-based paint provides the best shield from moisture. It is not, however, the most durable. It does not expand and contract as well as latex, and it is more prone to cracking. Oil/alkyd paints cure by air oxidation. These paints are low cost, with good durability, flexibility, and gloss retention. They are resistant to heat and solvents. Alkyd paints often contain lead pigments, known to cause numerous health hazards. The removal and disposal of lead paint is a regulated activity in all states.

Latex paint consists of a latex emulsion in water. Latex paint is often referred to as water-based paint. There are many types of latex paint, each formulated for a different application. They have excellent flexibility and color retention, with good adhesion, hardness, and resistance to chemicals.

2.1.7 Inspection Procedures for Timber

There are three basic procedures used to inspect a timber member. Depending on the type of inspection, the inspector may be required to use only one individual procedure or all procedures. They include:

- Visual
- Physical
- Advanced inspection techniques

Visual Examination

There are two types of visual inspections that may be required of an inspector. The first, called a cursory inspection, involves reviewing the previous inspection report and visually examining the members from beneath the bridge. A cursory inspection involves a visual assessment to identify obvious defects.

The second type of visual inspection is called a "hands-on" inspection. This type of visual inspection requires the inspector to visually assess all defective timber surfaces at a distance no further than an arm's length. The timber surfaces are given close visual attention to quantify and qualify any defects.

For timber members, visual inspections reveal areas that need further investigation such as checks, splits, shakes, fungus decay, deflection, or loose fasteners.

Physical Examination

Once the defects are identified visually, physical procedures must be used to verify the extent of the defect. Most physical inspection procedures for timber members involve destructive methods. An inspection hammer, on the other hand, does not and can be used to tap on areas and determine the extent of internal decay. This is done by listening to the sound the hammer makes. If it sounds hollow, internal decay may be present. Some methods or areas of physical examination include:

Pick or Penetration Test

A pick or penetration test involves lifting a small sliver of wood with a pick or pocketknife and observing whether or not it splinters or breaks abruptly. Sound wood splinters, while decayed wood breaks abruptly (see Figure 2.1.27).

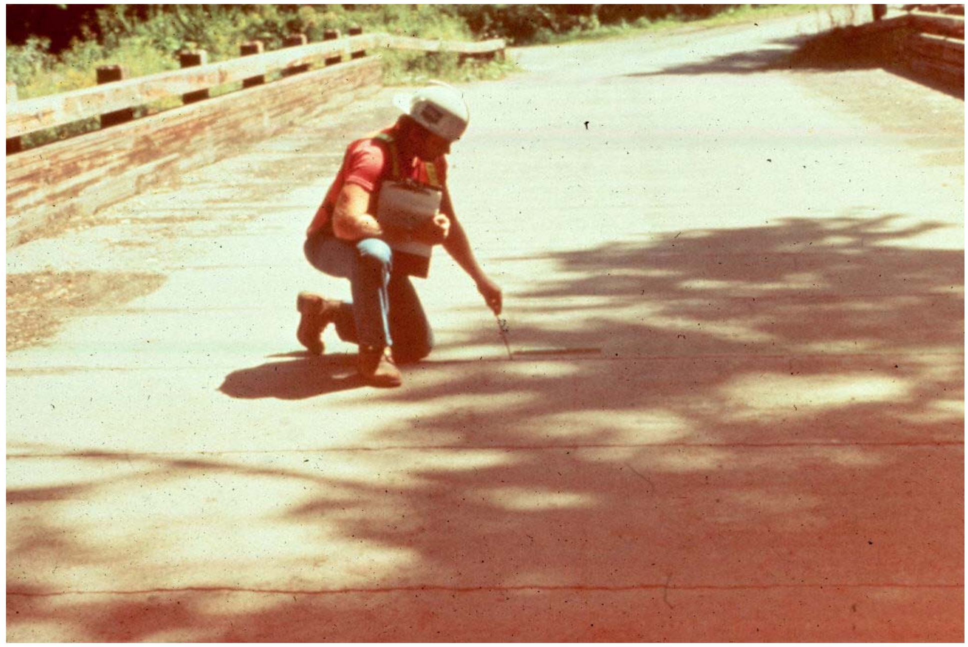

Figure 2.1.27 Inspector Probing Timber

Timber Boring and Drilling Locations

The following are common timber boring and drilling locations (see Figure 2.1.28):

- Deck planks - in the bottom, next to a beam.
- Beams - in sides near the deck and in the bottom over the bent cap.
- Cap - under the beams and over posts and piles.
- Post/pile - top under cap and bottom just above ground or water line.

An inspector may be required to take samples to determine the condition of the wood. When drilling or boring vertical faces, always drill at a slight upward angle so that any drainage will flow away from the plugged hole.

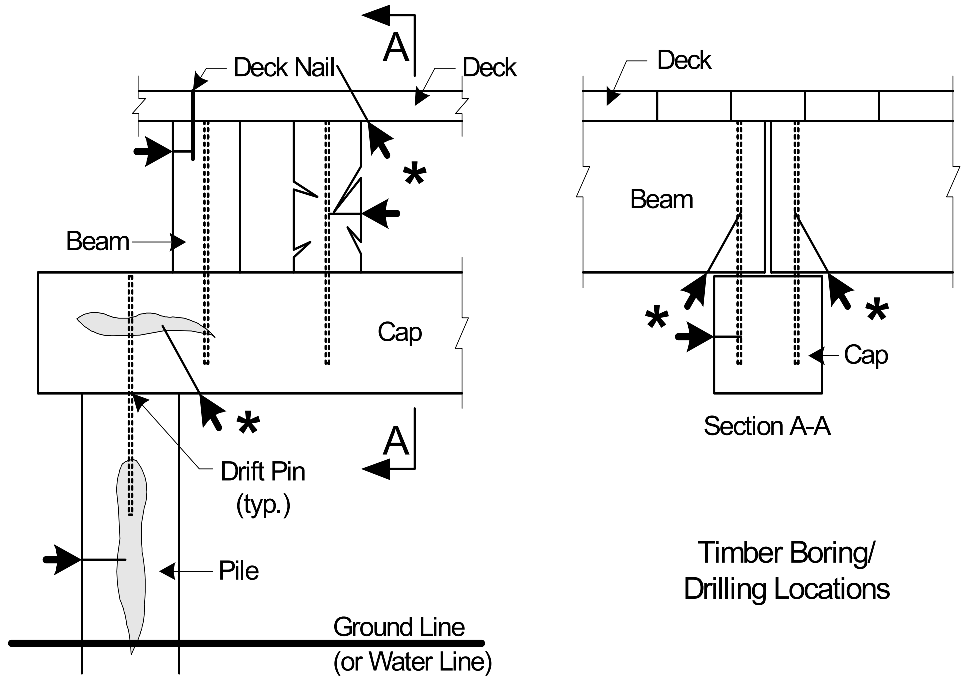

Figure 2.1.28 Timber Boring and Drilling Locations

Protective Coatings

When inspecting timber bridges, keep in mind the environment surrounding the bridge and how this can cause failures leading to rapid decay of the underlying wood members.

Paint Adhesion

Probe the paint with the point of a knife to test paint adhesion to wood. Attempt to lift the paint. Adhesion failure may occur between wood and paint or between layers of paint.

A more quantitative paint adhesion assessment is performed in accordance with American Society for Testing and Materials (ASTM) D-3359 "Measuring Adhesion by Tape Test". An "X" is cut through the paint to the wood surface. Adhesive test tape is applied over the "X" and removed in a continuous motion. The amount of paint (if any) removed is noted. Adhesion is rated on a scale of 0 to 5. Refer to ASTM D-3359 for the rating criteria.

Paint Dry Film Thickness

Paint dry film thickness is measured with a Tooke Gage (see Figure 2.1.26). With this instrument, a groove is cut at a known angle with the grain through the paint to expose the wood substrate. The thickness of each layer of paint is measured through a 50-power microscope built into the gage.

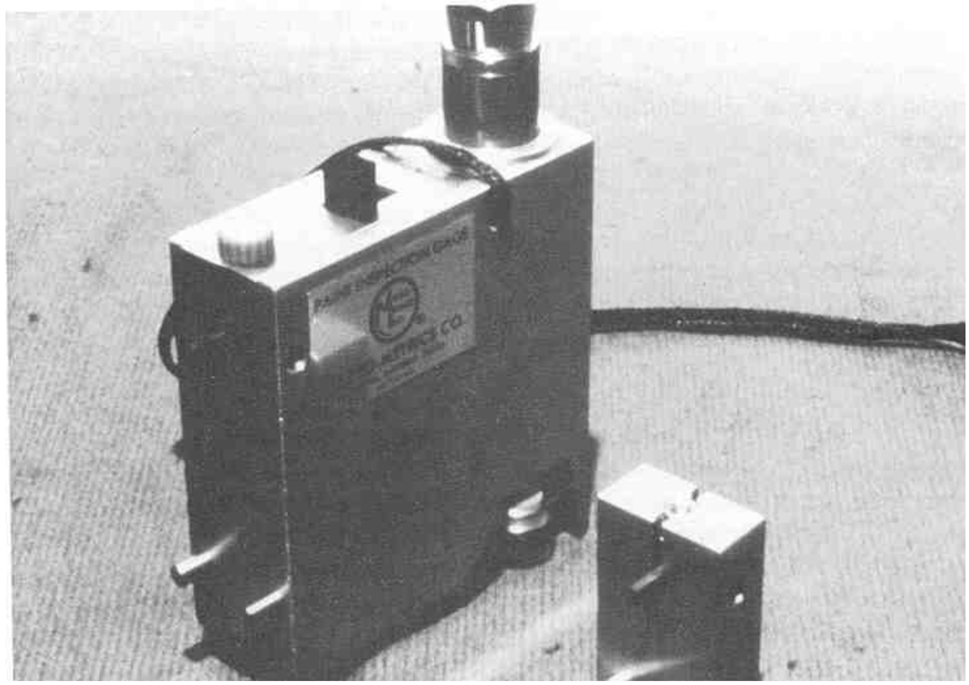

Figure 2.1.29 Tooke Gage Used to Measure Coating Dry Film Thickness

Repainting

If the coating is to be repainted, the type of paint in the existing topcoat must be known, since paints of different type may not adhere well to each other. Two methods to determine the type of existing paint are:

- Check historical records of previous painting
- Obtain paint samples from the bridge for laboratory analysis

Alternately, a test patch may be coated with new paint over intact existing paint. After the paint thoroughly dries in accordance with the manufacturer's specification, inspect the appearance and adhesion of the new paint.

Advanced Inspection Techniques

In addition, several advanced techniques are available for timber inspection. Nondestructive methods, described in Topic 13.1.2, include:

- Pol-Tek
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 13.1.3, include:

- Boring or drilling
- Moisture content
- Probing
- Shigometer

Detailed information about timber bridges can be found from the text, Timber Bridges, Design, Construction, Inspection and Maintenance, published by the USDA Forest Service. Latest information about timber bridge technology, including publications, can be obtained from the Forest Service's Timber Bridge Information Resource Center at (304) 285-1591 or at the website at http://www.fs.fed.us/. Information can also be obtained at the FHWA website, which is at TFHRC.dot.gov.

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Chapter 2
Bridge Materials

Topic 2.2 Concrete

2.2.1 Introduction

A large percentage of the bridge structures in the nation's highway network are constructed of reinforced concrete or prestressed concrete. It is important that the bridge inspector understand the basic characteristics of concrete in order to efficiently inspect and evaluate a concrete bridge structure.

Concrete, commonly mislabeled as "cement", is a mixture of various components that, when mixed together in the proper proportions, chemically react to form a strong durable construction material ideally suited for certain bridge components. Cement is only one of the basic ingredients of concrete. It is the "glue" that binds the other components together. Concrete is made up of the following basic ingredients:

- Portland cement
- Water
- Air
- Aggregates
- Admixtures (reducers, plasticizers, retarders)

Portland Cement

The first ingredient, Portland Cement, is one of the most common types of cement, and it is made with the following raw materials:

- Limestone - provides lime
- Quartz or cement rock - provides silica
- Claystone - provides aluminum oxide
- Iron ore - provides iron oxide

The cement is produced by placing the above materials through a three process high temperature kiln system. During the three process kiln system, the temperature can range from 100° C to 1510° C (212° F to 2750° F). The first zone in the kiln process is known as the drying process. During this process, the materials are dehydrated due to the high temperature. The calcining zone is the next step and results in the production of lime and magnesia. The final step, called the burning zone or clinkering zone, produces clinkers or nodules of the sintered materials. Upon cooling, the clinkers are ground into a powder and finish the Portland cement production process.

Water

The second ingredient, water, can be almost any potable water. Impurities in water, such as dissolved chemicals, salt, sugar, or algae, produce a variety of undesirable effects on the quality of the concrete mix. Therefore, water with a noticeable taste or odor may be suspect.

Air

The third ingredient of concrete is air. Small evenly distributed amounts of entrained air provide:

- Increased durability against freeze/thaw effects
- Reduced cracking
- Improved workability
- Reduced water segregation

Air entrainment also reduces the weight of concrete slightly. Many tiny air bubbles introduced into the plastic concrete naturally create lighter weight concrete. The typical air entrainment additive is a vinsol resin. Air entrainment additives act like dishwashing liquids. When mixed with water, they create bubbles. These bubbles become part of the concrete mix, creating tiny air voids. Through extensive lab testing, it has been proven that when exposed to freeze/thaw conditions, the voids prevent excess pressure buildup in the concrete.

Aggregates

The fourth ingredient, aggregates, comprise approximately 75% of a typical concrete mix by volume. Some aggregate qualities which result in a strong and durable concrete are:

- Abrasion resistance
- Weather resistance
- Chemical stability
- Chunky compact shape
- Smooth, non-porous surface texture
- Cleanliness and even gradation

Normal weight concrete has a unit weight of about 2240 to 2400 kg/m^3 (140 to 150 pcf). Typical aggregate materials for normal weight concrete are sand, gravel, crushed stone, and air-cooled, blast-furnace slag.

Lightweight concrete normally has a unit weight of 1200 to 1840 kg/m^3 (75 to 115 pcf). The weight reduction comes from the aggregates and air entrainment. Lightweight aggregates differ depending on the location where the lightweight concrete is being produced. The common factor in lightweight aggregates is that they all have many tiny air voids in them that make them lightweight with a low specific gravity.

Admixtures

The fifth ingredient of most concrete mixes is one or more admixtures to change the consistency, setting time, or concrete strength. Pozzolans are a common type of admixture used to reduce permeability. There are natural pozzolans such as diatomite and pumicite, along with artificial pozzolans which include admixtures such as fly ash.

Admixtures can either be minerals or chemicals. The mineral admixtures include fly ash, silica fume, and ground granulated blast-furnace slag. Chemical admixtures can include water reducers, plasticizers, retarders, high range water reducers, and superplasticizers.

Fly ash is a by-product from the burning of ground or powdered coal. Fly ash was added to concrete mixes as early as the 1930's. This turned out to be a viable way to dispose of fly ash while positively affecting the concrete. The use of fly ash in concrete mixes improves concrete workability, reduces segregation, bleeding, heat evolution and permeability, inhibits alkali-aggregate reaction, and enhances sulfate resistance.

The use of fly ash in concrete mixes also has some drawbacks, however, such as increased set time and reduced rate of strength gain in colder temperatures. Admixture effects are also reduced when fly ash is used in concrete mixes. This means, for example, that a higher percentage of air entrainment admixture is needed for concrete mixes using fly ash.

Silica fume (microsilica) results from the reduction of high purity quartz with coal in electric furnaces while producing silicon and ferrosilicon alloys. It affects concrete by improving compressive strength, bond strength, and abrasion resistance. Microsilica also reduces permeability. Concrete with a low permeability minimizes steel reinforcement corrosion, which is of major concern in areas where deicing agents are used. These properties have contributed to the increased use of high performance concrete in recent bridge design and construction.

Some disadvantages that result from the use of silica fume include a higher water demand in the concrete mix, a larger amount of air entraining admixture, and a decrease in workability.

Ground granulated blast-furnace slag is created when molten iron blast furnace slag is quickly cooled with water. This admixture can be substituted for cement on a 1:1 basis. However, it is usually limited to 25% in areas where the concrete will be exposed to deicing salts and to 50% in areas that do not need to use deicing salts.

Water reducing admixtures and plasticizers are used to aid workability at lower water/cement ratios, improve concrete quality and strength using less cement content, and help in placing concrete in adverse conditions. These admixtures can be salts and modifications of hydroxylized carboxylic acids, or modifications of lignosulfonic acids, and polymeric materials. Some of the potentially negative effects that are encountered when using water reducers and plasticizers include loss of slump and excess setting time.

Retarding admixtures are used to slow down the hydration process while not changing the long-term mechanical properties. This type of admixture is needed when heat is a problem. Retarders slow down the setting time to reduce unwanted temperature and shrinkage cracks which result from a fast curing mix.

2.2.2 Properties of Concrete

It is necessary for the bridge inspector to understand the different physical and mechanical properties of concrete and how they relate to concrete bridges in service today.

Physical Properties

The major physical properties of concrete are:

- Thermal expansion - concrete expands as temperature increases and contracts as temperature decreases
- Porosity - because of entrapped air, the cement paste never completely

fills the spaces between the aggregate particles, permitting absorption of water and the passage of water under pressure

- Volume changes due to moisture - concrete expands with an increase in moisture and contracts with a decrease in moisture
- Fire resistance - quality concrete is highly resistant to the effects of heat; however, temperatures over 370°C (700°F) may cause damage
- Formability - concrete can be cast to any shape prior to curing

Mechanical Properties

The major mechanical properties of concrete are:

- Strength - Plain, unreinforced concrete has a 28-day compressive strength ranging from about 17 MPa (2500 psi) to about 41 MPa (6000 psi). Higher strength concrete, with compressive strengths ranging from 41 MPa (6000 psi) to about 76 MPa (11,000 psi), is also available and becoming more commonly used. However, its tensile strength is only about 10% of its compressive strength, its shear strength is about 12% to 13% of its compressive strength, and its flexural strength is about 14% of its compressive strength (see Table 2.2.1).

  Six principal factors that increase concrete strength are:

  - Increased cement content
  - Increased aggregate strength
  - Decreased water-to-cement ratio
  - Decreased entrapped air
  - Increased curing time (extent of hydration)
  - Use of pozzolanic admixtures and slag

- Elasticity - Within the range of normal use, concrete is able to deform a limited amount under load and still return to its original orientation when the load is removed (elastic deformation). Elasticity varies as the square root of compressive strength. See Topic P.1 for modulus of elasticity and how it affects elastic deformation.

- Creep - In addition to elastic deformation, concrete exhibits long-term, irreversible, continuing deformation under application of a sustained load. Creep (plastic deformation) ranges from 100% to 200% of initial elastic deformation, depending on time.

- Isotropy - Plain, unreinforced concrete has the same mechanical properties regardless of which direction it is loaded.

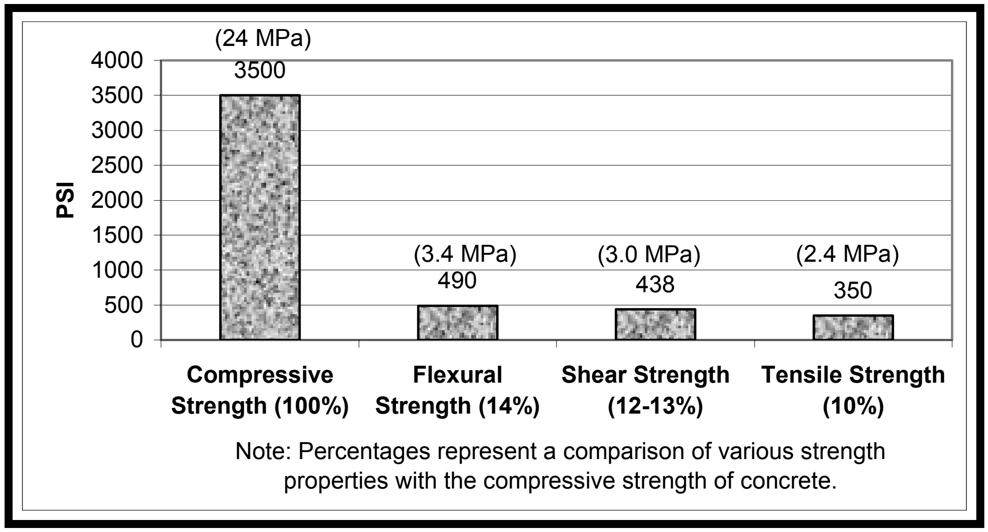


Table 2.2.1 Strength Properties of Concrete (24 Mpa) (3500 psi Concrete)

High Performance Concrete

High performance concrete (HPC) has been used for more than 20 years in the building industry. Under the FHWA's Strategic Highway Research Program (SHRP) Implementation Program, four types of high performance concrete mix designs were developed (see Table 2.2.2). High performance concrete is distinguished from regular concrete by its curing conditions and proportions of the ingredients in the mix design. The use of fly ash and high range water reducers play an important role in the design of HPC, as well as optimizing all components of the mix. Due to the increased strength and reduced permeability of HPC, bridge decks using HPC are expected to have double the life of conventional concrete bridge decks. The type and strength characteristics of concrete used to construct bridge components can be found in the bridge file under design specifications.

HPC Type	Minimum Strength Criteria	Water-Cementitious Ratio	Minimum Durability Factor
Very Early Strength (VES)	13.8 MPa (2,000 PSI)/ 6 hours	≤ 0.4	80%
High Early Strength (HES)	34.5 MPa (5,000 PSI)/ 24 hours	≤ 0.35	80%
Very High Strength (VHS)	69 MPa (10,000 PSI)/ 28 hours	≤ 0.35	80%
Fiber Reinforced	HES + (steel or poly)	≤ 0.35	80%
Additional information on the definition of HPC:			
- "HPC Defined for Highway Structures," Charles Goodspeed, Suneel Vanik			
Cook; *Concrete International*, February 1996, The American Concrete Ins			
- "Workshop Showcases High-Performance Concrete Bridges," *Focus* New			
May 1996.			

Table 2.2.2 FHWA's SHRP Implemented HPC Mix Designs

2.2.3 Reinforced Concrete

Concrete is commonly used in bridge applications due to its compressive strength properties. However, in order to supplement the limited tensile strength of concrete, tensile steel reinforcement is used (see Figure 2.2.1).

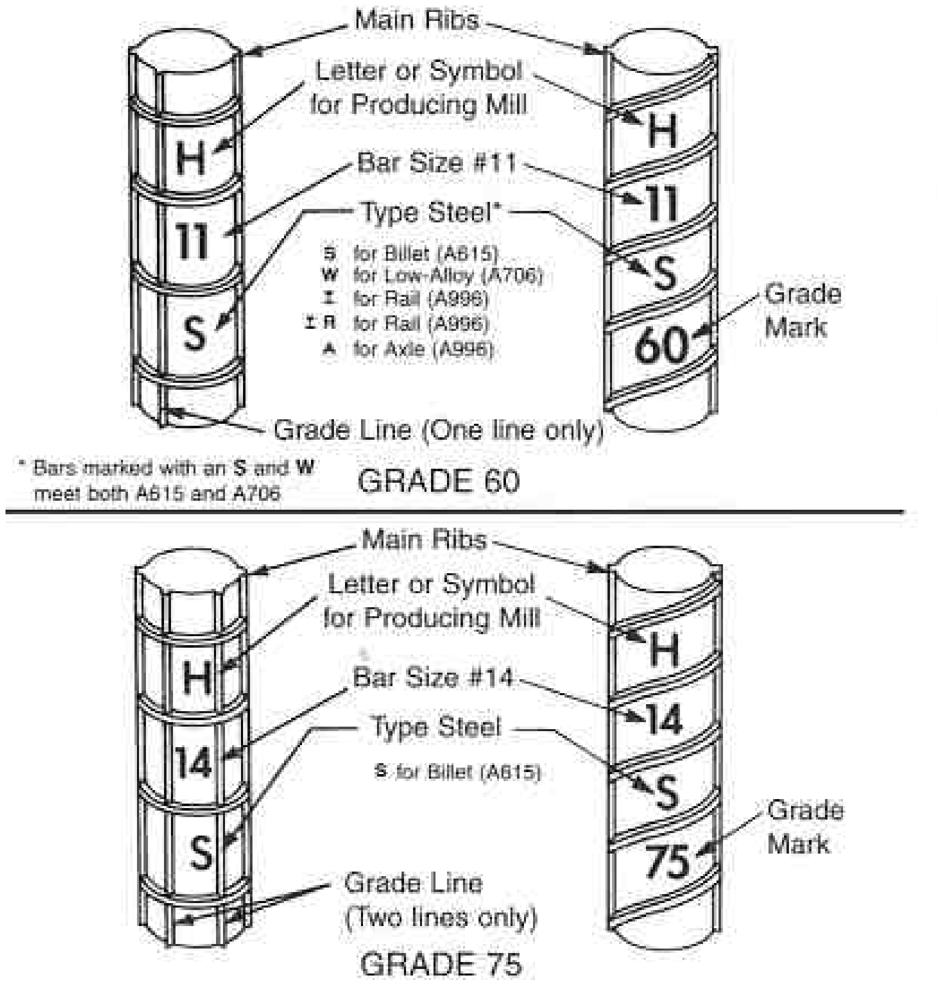
Figure 2.2.1 Concrete Member with Tensile Steel Reinforcement Showing

Steel reinforcement has a tensile yield strength of 276 MPa (40 ksi) or 414 MPa (60 ksi) and therefore has approximately 100 times the tensile strength of commonly used concrete. Therefore, in reinforced concrete members, the concrete resists the compressive forces and the steel reinforcement primarily resists the tensile forces. The type of steel reinforcement used in reinforced concrete is "mild steel", which is a term used for low carbon steels. The steel reinforcement is located close to the tension face of a structural member to maximize its efficiency.

Shear reinforcement is also needed to resist diagonal tension (refer to Topic P.1). Shear cracks start at the bottom of concrete members near the support and propagate upward and away from the support at approximately a 45° angle. Vertical or diagonal shear reinforcement is provided in this area to intercept the cracks and to stop the cracks from opening wider.

Reinforcing bars are also placed uniformly around the perimeter of a member to resist stresses resulting from temperature changes and volumetric changes of concrete. This steel is referred to as temperature and shrinkage steel.

Steel reinforcing bars can be "plain" or smooth surfaced, or they can be "deformed" with a raised gripping pattern protruding from the surface of the bar (see Figure 2.2.2). The gripping pattern improves bond with the surrounding concrete. Modern reinforced concrete bridges are generally constructed with "deformed" reinforcing steel.



INCH-POUND BAR SIZE	DIAMETER (in.)	METRIC BAR SIZE	DIAMETER (mm)
#3	0.375	#10	9.5
#4	0.500	#13	12.7
#5	0.625	#16	15.9
#6	0.750	#19	19.1
#7	0.875	#22	22.2
#8	1.000	#25	25.4
#9	1.128	#29	28.7
#10	1.270	#32	32.3
#11	1.410	#36	35.8
#14	1.693	#43	43.0
#18	2.257	#57	57.3

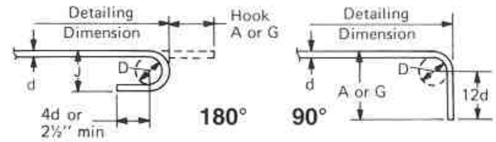
Figure 2.2.2 Standard Deformed Reinforcing Bars (Source: Concrete Reinforcing Steel Institute)

STANDARD HOOK DETAILS

in accordance with ACI 318-99

All grades of steel (min yield strengths)
D = Finished inside bend diameter
d = Bar diameter

D = 6d for #3 through #8
D = 8d for #9, #10 and #11
D = 10d for #14 and #18



RECOMMENDED END HOOKS

BAR SIZE	D	180° HOOKS		90° HOOKS
		A or G	J	A or G
#3	2¼"	5"	3"	6"
#4	3"	6"	4"	8"
#5	3¾"	7"	5"	10"
#6	4½"	8"	6"	1'-0"
#7	5¼"	10"	7"	1'-2"
#8	6"	11"	8"	1'-4"
#9	9½"	1'-3"	11¾"	1'-7"
#10	10¾"	1'-5"	1'-1¼"	1'-10"
#11	12"	1'-7"	1'-2¾"	2'-0"
#14	18¼"	2'-3"	1'-9¾"	2'-7"
#18	24"	3'-0"	2'-4½"	3'-5"

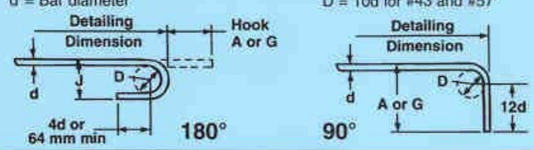
STEEL TYPE	BAR SIZE RANGE	GRADE	MINIMUM YIELD, KSI	MINIMUM TENSILE, KSI
Billet A615	#3 - #6	40	40	70
	#3 - #18	60	60	90
	#6 - #18	75	75	100
Low-Alloy A706	#3 - #18	60	60	80
Rail & Axle A996	#3 - #8	40	40	70
	#3 - #8	50	50	80
	#3 - #8	60	60	90

STANDARD METRIC HOOK DETAILS

in accordance with ACI 318M-99

All grades of steel (min yield strengths)
D = Finished inside bend diameter
d = Bar diameter

D = 6d for #10 through #25
D = 8d for #29, #32 and #36
D = 10d for #43 and #57



RECOMMENDED END HOOKS

BAR SIZE	D	180° HOOKS		90° HOOKS
		A or G	J	A or G
#10	60	125	80	150
#13	80	150	105	200
#16	95	175	130	250
#19	115	200	155	300
#22	135	250	180	375
#25	155	275	205	425
#29	240	375	300	475
#32	275	425	335	550
#36	305	475	375	600
#43	465	675	550	775
#57	610	925	725	1050

NOTE: All dimensions are in millimeters (mm).

STEEL TYPE	BAR SIZE RANGE	GRADE	MINIMUM YIELD, MPa	MINIMUM TENSILE, MPa
Billet A615M	#10 - #19	300	300	500
	#10 - #57	420	420	620
	#19 - #57	520	520	690
Low-Alloy A706M	#10 - #57	420	420	550
Rail & Axle A996	#10 - #25	300	300	500
	#10 - #25	350	350	550
	#10 - #25	420	420	620

Figure 2.2.2 Standard Deformed Reinforcing Bars (Source: Concrete Reinforcing Steel Institute) (Continued)

In US units, reinforcing bars up to 1” nominal diameter are identified by numbers that correspond to their nominal diameter in eighths of an inch. For example, a #4 bar has a 1/2 inch nominal diameter (or 4 times 1/8 inch). For the remaining bar sizes (#9, #10, #11, #14, and #18), the area is equivalent to the old 1”, 1 1/8”, 1 1/4”, 1 1/2”, and 2” square bars, respectively.

Reinforcing bars can also be used to increase the compressive strength of a concrete member. When reinforcing bars are properly cast into a concrete member, the steel and concrete acting together provide a strong, durable construction material.

Reinforcing bars can be protected or unprotected from corrosion. Unprotected reinforcement is referred to as “black” steel because only mill scale is present on the surface. Unprotected reinforcement is primarily used in concrete footings or other concrete members that are underground or not exposed to moisture.

The deformed epoxy coated bar is the most common type of protected reinforcing bar used. It is commonly specified when a concrete member may be exposed to an adverse environment. The epoxy provides a protective coating against corrosion agents such as de-icing chemicals and brackish water, and is inexpensive compared to other protective coatings. Another type of protected reinforcing bar is the galvanized bar. Unprotected bars are given a zinc coating, which slows down or stops the corrosion process. See Topic 2.2.9 for a detailed description of reinforcement protective coatings.

In the near future, tensile reinforcement made of Fiber Reinforced Polymer (FRP) composites may become common. Currently the FHWA and other government agencies and universities are performing research to better understand the properties and appropriate methods for use of this material in new construction. FRP is lighter weight than traditional steel reinforcement, can be designed with a wide range of mechanical properties including tensile, flexural, impact and compressive strengths and provides a viable alternative in areas where deicing salts are used due to the fact that the deck does not deteriorate due to steel reinforcement corrosion. Current holdbacks to widespread use of FRP reinforcement are the relative cost when compared to steel, the limited amount of experience contractors have building with it, and not having much performance data.

2.2.4 Prestressed Concrete

Another type of concrete used in bridge applications is prestressed concrete, which uses high tensile strength steel strands as reinforcement. To reduce the tensile forces in a concrete member, internal compressive forces are induced through prestressing steel tendons or strands. When loads are applied to the member, any tensile forces developed are counterbalanced by the internal compressive forces induced by the prestressing steel. By prestressing the concrete in this manner, the final tensile forces are typically within the tensile strength limits of plain concrete. Therefore, properly designed prestressed concrete members do not develop flexure cracks under service loads (see Figure 2.2.3).

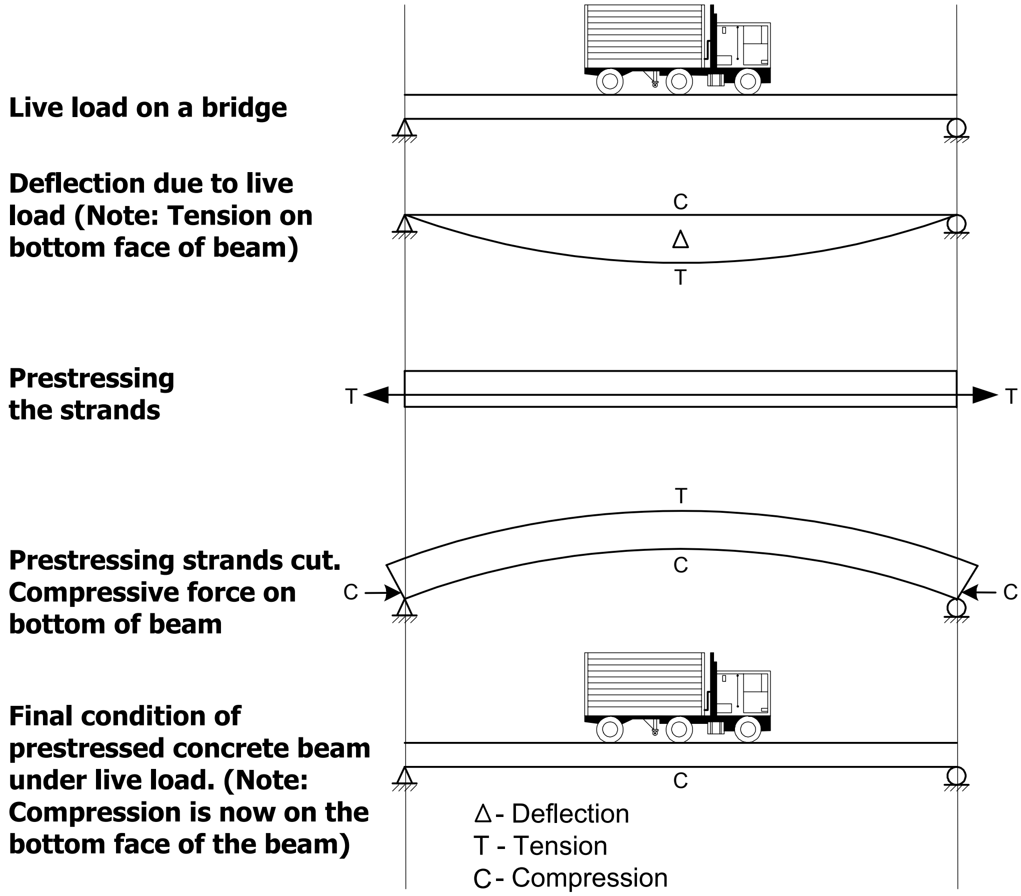


Figure 2.2.3 Prestressed Concrete Beam

There are three methods of prestressing concrete:

- Pretensioning - during fabrication of the member, prestressing steel is placed and tensioned prior to casting and curing of the concrete (see Figure 2.2.4)
- Post-tensioning - during fabrication of the member, ducts are cast-in-place so that after curing, the prestressing steel can be passed through the ducts and tensioned (see Figure 2.2.5)
- Combination method - this is used for long members for which the required prestressing force cannot safely be applied using pretensioning only

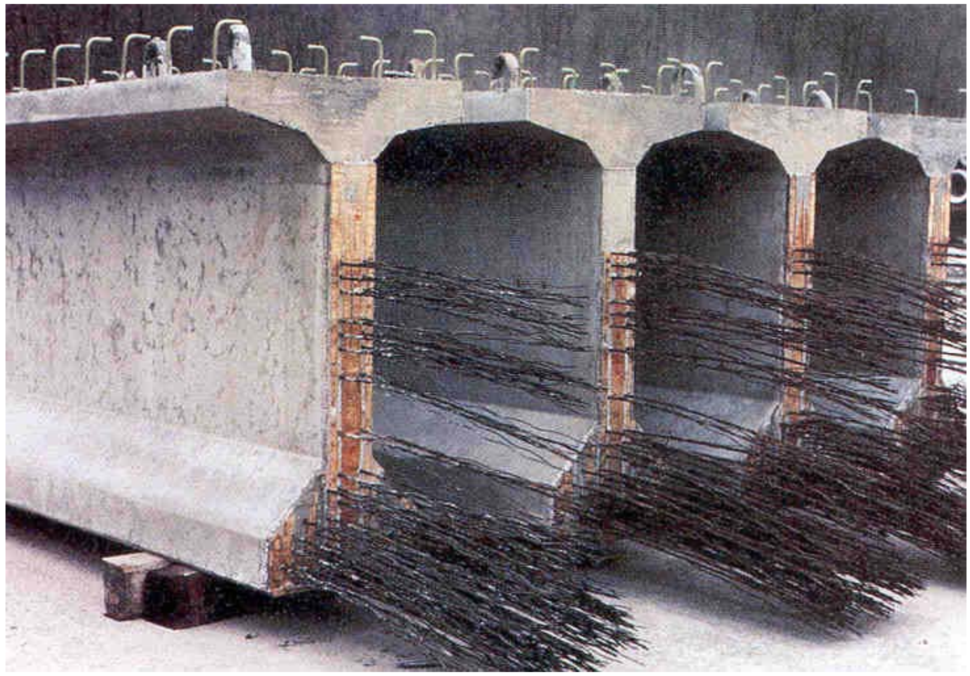


Figure 2.2.4 Pretensioned Concrete I-beams

Figure 2.2.5 Post-tensioned Concrete Box Girder

Steel for prestressing, which is named high tensile strength steel, comes in three basic forms:

- Wires (ASTM A421) - single wires or parallel wire cables; the parallel wire cables are commonly used in post-tensioning operations; the most popular wire size is 6 mm (1/4 inch) diameter and the most common grade of steel is the 1860 Mpa (270 ksi) grade.

- Strands (ASTM A416) - fabricated by twisting wires together; the seven wire strand is the most common type of prestressing steel used in the United States, and the 1860 Mpa (270 ksi) grade is most commonly used today

- Bars (ASTM A322 and A29) - high tensile strength bars typically have a minimum ultimate stress of 1000 Mpa (145 ksi); the bars have full length deformations that also serve as threads to receive couplers and anchorage hardware

Epoxy coated prestressing strand is a newer alternative to help minimize the amount of corrosion that occurs to otherwise unprotected strands. The epoxy is applied to the ordinary seven wire low relaxation prestressing strand through a process called "fusion bonding". Once the epoxy is applied, the strand has very little bond capacity and an aluminum oxide grit has to be applied to aid in the bonding. From recent testing by the FHWA, the epoxy coated strands have a tendency to slip when advanced curing temperatures are 63°C (145°F) and above. This slip occurs because the epoxy material begins to melt at these temperatures. Because the epoxy coating has a tendency to melt, this type of alternative is not used unless protection of the prestressing strand is critical.

In pretensioned members, transfer of tendon tensile stress occurs through bonding, which is the secure interaction of the prestressing steel with the surrounding concrete. This is accomplished by casting the concrete in direct contact with the prestressed steel.

In post-tensioned members, transfer of tendon tensile stress is accomplished by mechanical end anchorages and locking devices. If bonding is also desired, special ducts are used which are pressure injected with grout after the tendons are tensioned and locked off.

For purposes of crack control in end sections of pretensioned members, the prestressing steel is sometimes debonded. This is accomplished by providing a protective cover on the steel, preventing it from contacting the concrete.

For post-tensioned members, when bonding is not desirable, grouting of tendon ducts is not performed and corrosion protection in the form of galvanizing, greasing, or some other means must be provided.

In prestressed concrete beams, shear strength is enhanced by the local compressive stress present. However, mild shear reinforcement is still required. Similar to reinforced concrete, prestressed concrete also requires mild steel temperature and shrinkage reinforcement.

2.2.5 Types of Concrete Deterioration

In order to properly inspect a concrete bridge, the inspector must be able to recognize the various types of defects or deterioration associated with concrete. The inspector must also understand the causes of the defects or deterioration and how to examine them. There are many common defects or deterioration that occur on reinforced concrete bridges:

- Cracking (flexure, shear, freeze-thaw cycles)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Efflorescence
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage

Structural Cracks

A crack is a linear fracture in concrete. It may extend partially or completely through the member. There are two basic types of cracks: structural and non-structural cracks. Structural cracks are caused by dead load and live load stresses. Cracking is considered normal for mildly reinforced concrete (e.g., in cast-in-place tee-beams) as long as the cracks are small and there are no rust stains or other signs of deterioration present. Larger structural cracks indicate potentially serious problems, because they are directly related to the structural capacity of the member. When cracks can be observed opening and closing under load, they are referred to as “working” cracks. See Table 2.2.3 for crack width guidelines. There are two types of structural cracks: flexure and shear (see Figure 2.2.6).

Flexure Cracks

Flexure cracks are caused by tensile forces and therefore develop in the tension zones. Tension zones occur either on the bottom or the top of a member, depending on the span configuration. Tension zones can also occur in substructure components. Tension cracks terminate when they approach the neutral axis of the member. If a beam is a simple span structure (refer to Topic P.1.9), flexure cracks can often be found at the mid-span at the bottom of the member where bending or flexure stress is greatest (see Figure 2.2.7). If the beams are continuous span structures (refer to Topic P.1.9), flexure cracks occur at the top of members at or near their supports.

Shear Cracks

Shear cracks are caused by diagonal tensile forces that typically occur in the web of a member near the supports where shear stress is the greatest. Normally, these cracks initiate near the bearing area, beginning at the bottom of the member, and extending diagonally upward toward the center of the member (see Figure 2.2.8). Shear cracks also occur in abutment backwalls, stems and footings, pier caps, columns, and footings.

Although structural cracks are typically caused by dead load and live load forces, they can also be caused by overstresses in members resulting from unexpected secondary forces. Restricted thermal expansion or contraction such as caused by frozen bearings, or forces due to the expansion of an approach slab or failure of a backwall can induce significant forces which result in cracks (see Figure 2.2.9).

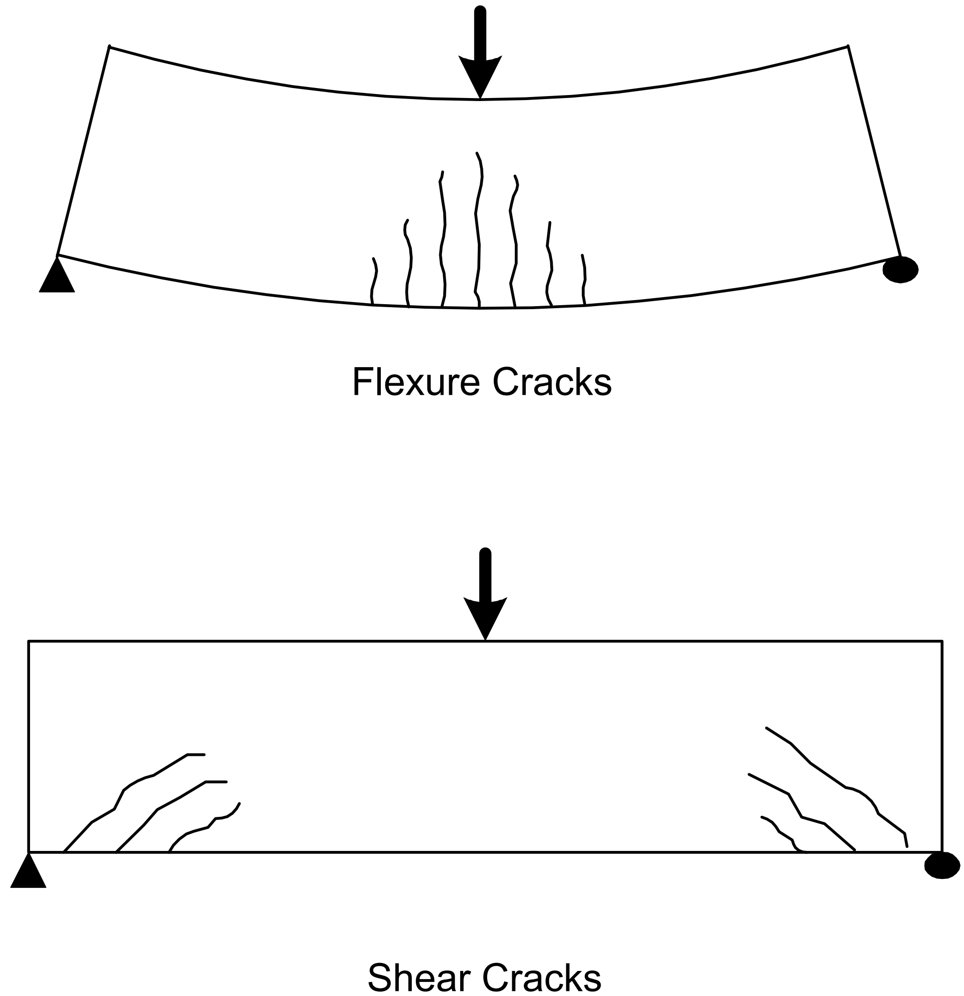


Figure 2.2.6 Structural Cracks

Figure 2.2.7 Flexural Crack on a Tee Beam

Figure 2.2.8 Shear Crack on a Slab Beam

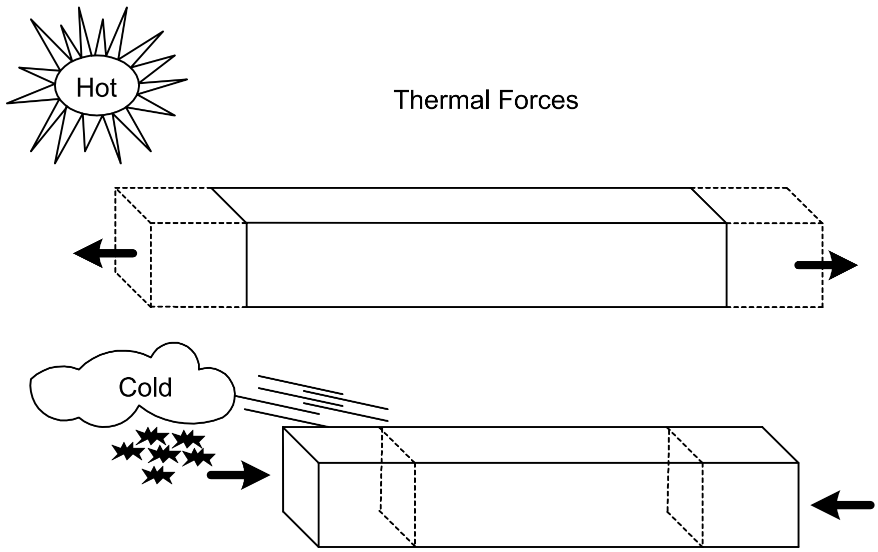


Figure 2.2.9 Thermal Forces

Crack Size

Crack size is very important in assessing the condition of an in-service bridge. Cracks may extend partially or completely through the concrete member. On reinforced concrete, cracking will usually be large enough to be seen with the naked eye. A crack comparitor card can be used to measure and differentiate cracks (see Figure 2.2.10 and Table 2.2.3).

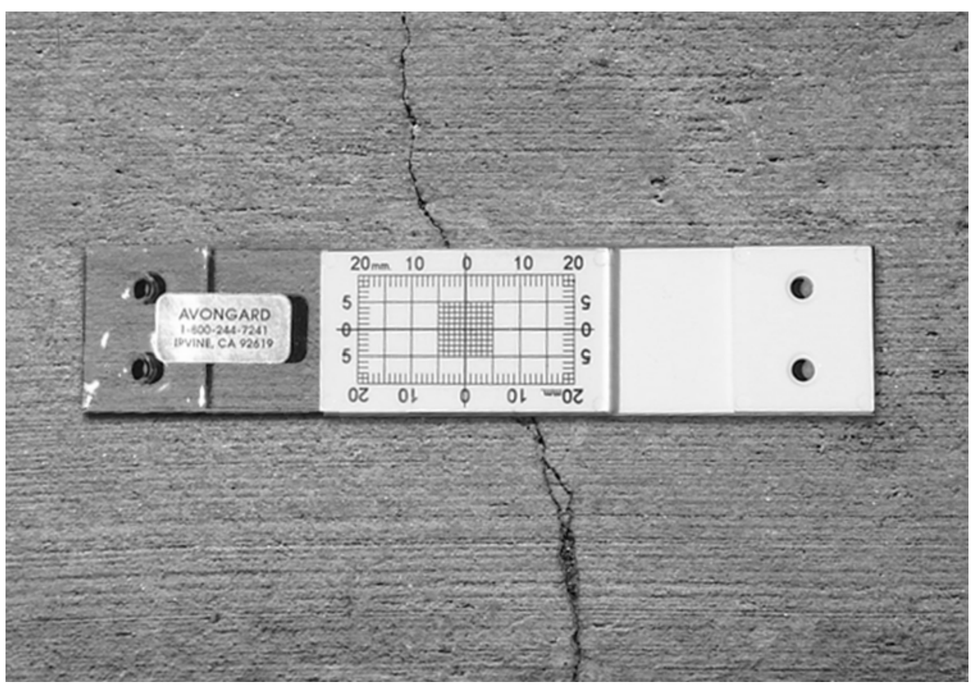


Figure 2.2.10 Crack Comparitor Card and Crack Gauge

CRACK WIDTH GUIDELINES				
	REINFORCED CONCRETE		PRESTRESSED CONCRETE	
	English	Metric	English	Metric
HAIRLINE (HL)	< 1/16"	< 1.6mm	< 0.004"	< 0.1mm
	(0.0625)			
NARROW (N)	1/16" to 1/8"	1.6 to 3.2mm	0.004 to 0.009"	0.1 to 0.23mm
	0.0625" – 0.125"			
MEDIUM (M)	1/8" to 3/16"	3.2 to 4.8mm	0.010 to 0.030"	0.25 to 0.76mm
	0.125" – 0.1875"			
WIDE (W)	>3/16"	> 4.8mm	> 0.030"	> 0.76mm
	>0.1875"			

Table 2.2.3 Crack Width Guidelines

Cracks can be classified as hairline, narrow, medium, or wide. Hairline cracks are small and cannot be measured with normal equipment, such as a six-foot rule. Medium and wide cracks are cracks that can be measured by simple means or a crack comparitor card. On conventionally reinforced structures, hairline cracks are usually insignificant. All other crack widths may be significant and should be monitored and recorded in the inspection notes.

On prestressed structures, all cracks are significant and an optical crack gauge is the proper instrument needed to measure and differentiate cracks.

When reporting cracks, the length, width, location, and orientation (horizontal, vertical, or diagonal) should be noted. Both large and small cracks in main members, especially in prestressed members, should be carefully recorded. The presence of rust stains or efflorescence or evidence of differential movement on either side of the crack should be indicated.

Nonstructural Cracks

Nonstructural cracks result from internal stresses due to dimensional changes. Nonstructural cracks are divided into three categories:

- Temperature cracks (see Figure 2.2.11)
- Shrinkage cracks (see Figure 2.2.12)
- Mass concrete cracks

Though these cracks are nonstructural and relatively small in size, they provide openings for water and contaminants, which can lead to serious problems. Temperature cracks are caused by the thermal expansion and contraction of the concrete. Concrete expands or contracts as its temperature rises or falls. If the concrete is prevented from contracting, due to friction or because it is being held in place, it will crack under tension. Inoperative bearing devices and clogged expansion joints can also cause this to occur. Shrinkage cracks are due to the shrinkage of concrete caused by the curing process. Volume reduction due to curing is also referred to as plastic shrinkage. Plastic shrinkage cracks occur while the concrete is still plastic and are usually short, irregular shapes and do not extend the full depth into the member. Mass concrete cracks occur due to thermal gradients (differences between interior and exterior) in massive sections immediately after placement and for a period of time thereafter. Temperature, shrinkage, and mass concrete cracks typically do not significantly affect the structural strength of a concrete member.

Figure 2.2.11 Temperature Cracks

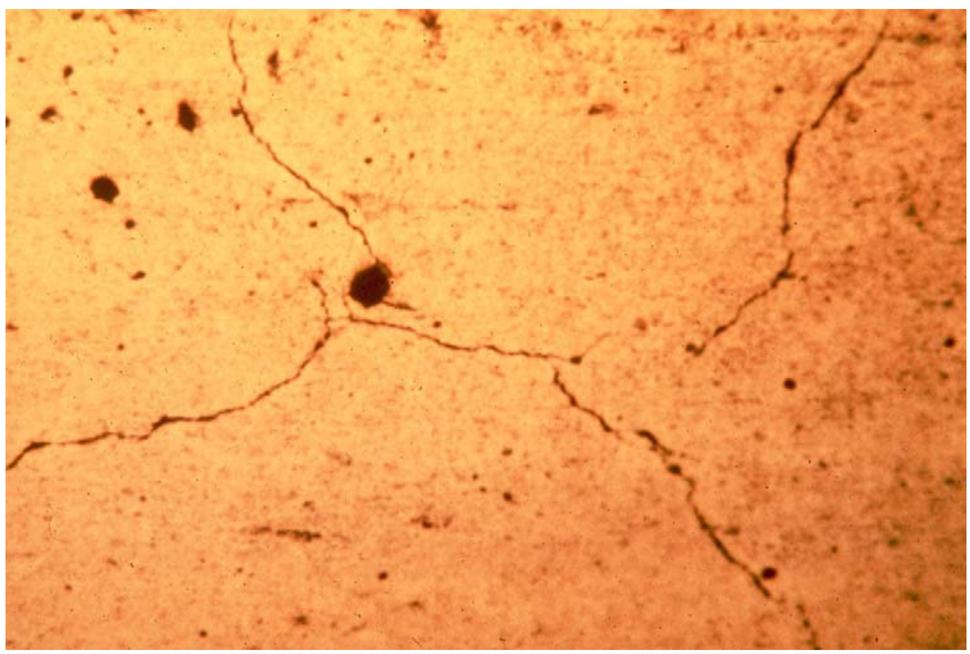


Figure 2.2.12 Shrinkage Cracks

In concrete bridge decks, temperature and shrinkage cracks can occur in both the transverse and longitudinal directions. In retaining walls and abutments, these cracks are usually vertical, and in concrete beams, these cracks occur vertically or transversely on the member. However, since temperature and shrinkage stresses exist in all directions, the cracks could have other orientations.

Inspectors must exercise care in distinguishing between nonstructural cracks and structural cracks. However, regardless of the crack type, water seeps in and causes the reinforcement to corrode. The corroded reinforcement expands and exerts pressure on the concrete. This pressure can cause delaminations and spalls.

Crack Orientation

In addition to classifying cracks as either structural or nonstructural and recording their lengths and widths, inspectors must also describe the orientation of the cracks. The orientation of the crack with respect to the loads and supporting members is an important feature that must be recorded accurately to ensure the proper evaluation of the crack. The orientation of cracks may generally be described by one of the following five categories:

- Transverse cracks – These are fairly straight cracks that are roughly perpendicular to the centerline of the bridge or a bridge member (see Figure 2.2.13).

- Longitudinal cracks - These are fairly straight cracks that run parallel to the centerline of the bridge or a bridge member (see Figure 2.2.14).

- Diagonal cracks - These cracks are skewed (at an angle) to the centerline of the bridge or a bridge member, either vertically or horizontally.

- Pattern or map cracking - These are inter-connected cracks that form networks of varying size. They vary in width from barely visible, fine cracks to cracks with a well defined opening. Map cracking resembles the lines on a road map (see Figure 2.2.15).

- Random cracks - These are meandering, irregular cracks. They have no particular form and do not logically fall into any of the types described above.

Figure 2.2.13 Transverse Cracks

Figure 2.2.14 Longitudinal Cracks

Figure 2.2.15 Pattern or Map Cracks

Scaling

Scaling is the gradual and continuing loss of surface mortar and aggregate over an area due to the chemical breakdown of the cement bond. Scaling is accelerated when the member is exposed to a harsh environment. Scaling is classified in the following four categories:

- Light or minor scale - loss of surface mortar up to 6 mm (¼ inch) deep, with surface exposure of coarse aggregates (see Figure 2.2.16)
- Medium or moderate scale - loss of surface mortar from 6 to 13 mm (¼ inch to ½ inch) deep, with mortar loss between the coarse aggregates (see Figure 2.2.17)
- Heavy scale - loss of surface mortar from 13 to 25 mm (½ inch to 1 inch) deep; coarse aggregates are clearly exposed (see Figure 2.2.18)
- Severe scale - loss of coarse aggregate particles, as well as surface mortar and the mortar surrounding the aggregates; depth of the loss exceeds 25 mm (1 inch); reinforcing steel is usually exposed (see Figure 2.2.19)

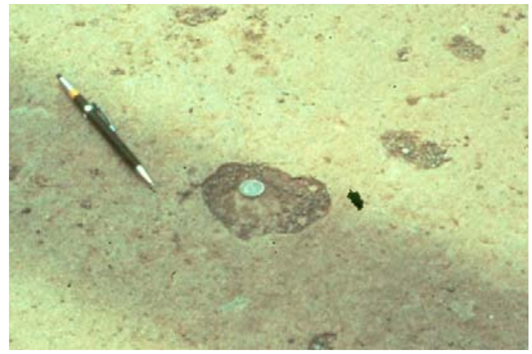


Figure 2.2.16 Light or Minor Scale

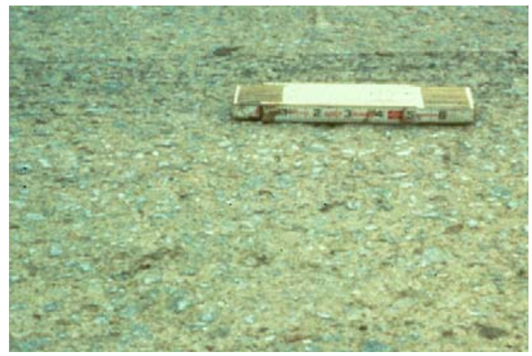


Figure 2.2.17 Medium or Moderate Scale

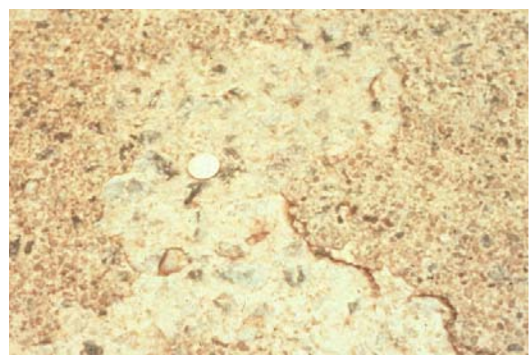


Figure 2.2.18 Heavy Scale

Figure 2.2.19 Severe Scale

When reporting scaling, the inspector should note the location of the defect, the size of the affected area, and the scaling classification. For severe scale, the depth of penetration of the defect should also be recorded.

Delamination

Delamination occurs when layers of concrete separate at or near the level of the top or outermost layer of reinforcing steel. The major cause of delamination is expansion of corroding reinforcing steel. This is commonly caused by intrusion of chlorides or salt. Another cause of delamination is severe overstress in a member. Delaminated areas give off a hollow "clacking" sound when tapped with a hammer. When a delaminated area completely separates from the member, the resulting depression is called a spall.

When reporting delamination, the inspector should note the location and the size of the defect.

Spalling

A spall is a depression in the concrete (see Figure 2.2.20). Spalls result from the separation and removal of a portion of the surface concrete, revealing a fracture roughly parallel to the surface. Spalls can be caused by corroding reinforcement, friction from thermal movement, and overstress. Reinforcing steel is often exposed in a spall, and the common shallow pothole in a concrete deck is considered a spall. Spalls are classified as follows:

- Small spalls - not more than 25 mm (1 inch) deep or approximately 150 mm (6 inches) in diameter
- Large spalls - more than 25 mm (1 inch) deep or greater than 150 mm (6 inches) in diameter

Figure 2.2.20 Spalling on a Concrete Deck

When concrete is overstressed, it gives or fractures. Over time, the fracture opens wider from debris, freeze/thaw cycles, or more overstress. This cycle continues until a spall is formed. Spalls caused from overstress are very serious and should be brought to the attention of the Chief Bridge Engineer. Most spalls are caused from corroding reinforcement, but if the spall is located at or near a high moment region, overstress may be the cause. Examples that might indicate a spall was caused by overstress include:

- A spall that is at or near flexure cracks in the lower portion of a beam at mid-span
- A spall that is at or near flexure cracks in the top of a continuous member over a support

Similarly, when concrete is overstressed in compression, it is common for the surface to spall.

When reporting spalls, the inspector should note the location of the defect, the size of the area, and the depth of the defect.

Chloride Contamination

Chloride contamination in concrete is the presence of recrystalized soluble salts. Concrete is exposed to chlorides in the form of deicing salts, acid rain, and in some cases, contaminated water used in the concrete mix. It causes accelerated reinforcement corrosion that leads to cracking of the concrete.

Efflorescence

The presence of cracks permits moisture absorption and increased flow within the concrete that is evidenced by dirty-white surface deposits called efflorescence. Efflorescence is a combination of calcium carbonate leached out of the cement paste and other recrystalized carbonate and chloride compounds (see Figure 2.2.21). In order to estimate the percent of concrete contaminated by chloride, nondestructive testing is required (refer to Topic 13.2.2).

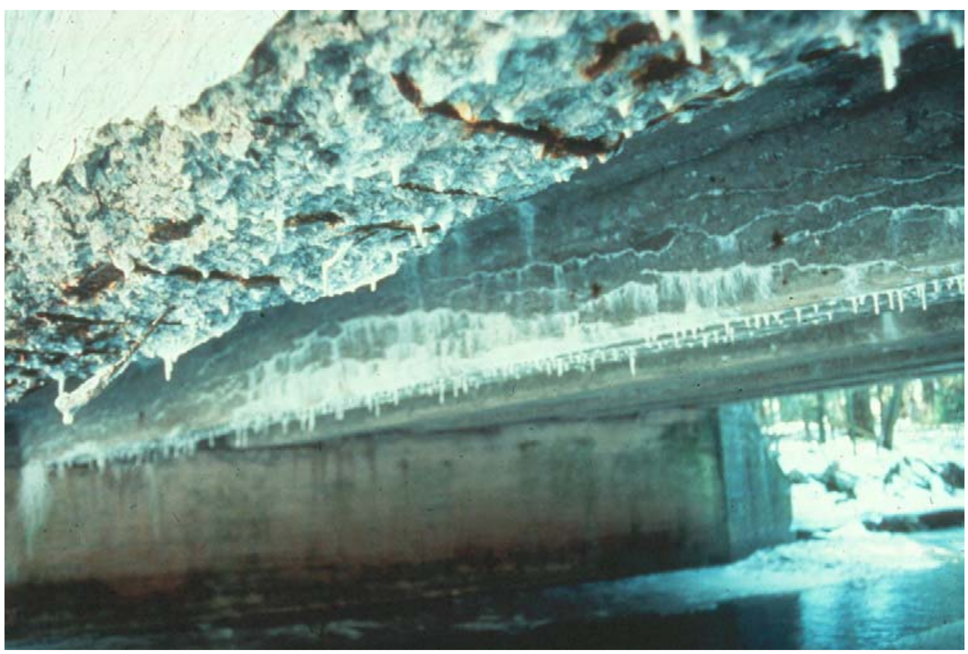


Figure 2.2.21 Efflorescence

Ettringite Formation

Ettringite formation is an internal defect that occurs in concrete from the reaction of sulfates, calcium aluminates, and water. From this reaction, ettringite, which is a crystalline mineral, expands up to eight times in volume compared to the volume of the tricalcium nitrates (C_3A). Ettringite formation is initially formed when water is added to the cement but prior to the concrete's initial set. The initial formation does not harm the concrete. A secondary or delayed ettringite formation occurs after the concrete has hardened. This formation creates very high forces in hardened concrete and is the cause of the deterioration. The only way to identify ettringite formation as a cause of premature concrete deterioration is through advanced inspection techniques such as petrographic analysis. Recent studies have shown that ettringite formation is linked to alkali-silica reaction (ASR), but further research is still needed.

Honeycombs

Honeycombs or construction voids are hollow spaces or voids that may be present within the concrete. Honeycombs are construction defects caused by improper vibration during concrete placement, resulting in the segregation of the coarse aggregates from the fine aggregates and cement paste. In some cases, honeycombs are the result of insufficient vibration, where the entire concrete mix does not physically reach the form surface (see Figure 2.2.22).

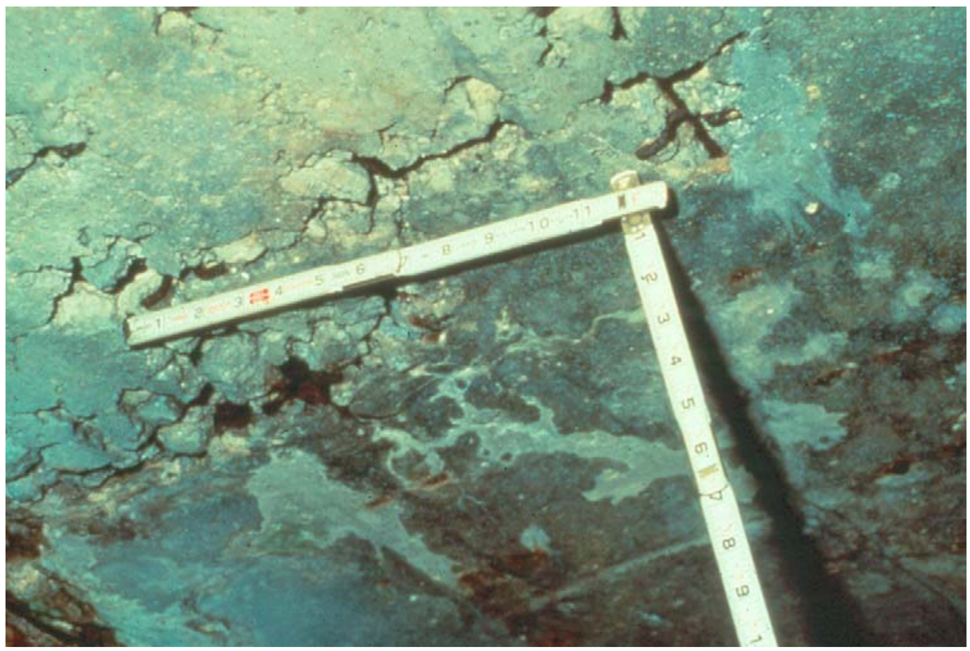


Figure 2.2.22 Honeycomb

Pop-outs

Pop-outs are conical fragments that break out of the surface of the concrete, leaving small holes. Generally, a shattered aggregate particle will be found at the bottom of the hole, with a part of the fragment still adhering to the small end of the pop-out cone. Pop-outs are caused by aggregates which expand with absorption of moisture. Other causes of pop-outs include use of reactive aggregates and high alkali cement.

Wear

Wear is the gradual removal of surface mortar due to friction and occurs to concrete surfaces, like a bridge deck, when exposed to traffic. Advanced wear exhibits polished aggregate, which is potentially a safety hazard when the deck is wet. The scraping action of snowplows and street sweepers also wears the deck surface and damages curbs, parapets, and pier faces.

Collision Damage

Trucks, derailed railroad cars, or marine traffic may strike and damage concrete bridge components (see Figures 2.2.23 and 2.2.24). The damage is generally in the form of cracking or spalling, with exposed reinforcement. Prestressed beams are particularly sensitive to collision damage, as exposed tendons undergo stress corrosion and fail prematurely.

Figure 2.2.23 Concrete Column Collision Damage

Figure 2.2.24 Collision Damage to Prestressed Concrete I-Beam

Abrasion

Abrasion damage is the result of external forces acting on the surface of the concrete member and is similar to wear. Erosive action of silt-laden water running over a concrete surface and ice flow in rivers and streams can cause considerable abrasion damage to concrete piers and pilings. In addition, concrete surfaces in surf zones may be damaged by the abrasive action of sand and silt in the water. Abrasion damage can be accelerated by freeze-thaw cycles. This will usually occur near the water line on concrete piers. The use of the term "scour" to indicate "abrasion" is incorrect. The term scour is used to describe the loss of streambed material from around the base of a pier or abutment due to stream flow or tidal action.

Overload Damage

Overload damage or serious structural cracking occurs when concrete members are sufficiently overstressed. Concrete decks, beams, and girders are all subject to damage from such overload conditions. Note any excessive vibration or deflection that may occur under traffic, which can indicate overstress. Other visual signs that can indicate overstress include excessive sagging, spalling, and/or cracking at the mid-span of simple span structures and at the supports of continuous span structures. Permanent deformation is another visual sign of overstress damage in a member. If overload damage is detected or suspected, the Chief Bridge Engineer should be notified immediately.

2.2.6 Reinforcing Steel Corrosion

Due to the chemistry of the concrete mix, reinforcing steel embedded in concrete is normally protected from corrosion. In the high alkaline environment of the concrete, a tightly adhering film forms on the steel that protects it from corrosion. However, this protection is eliminated by the intrusion of chlorides, which enables water and oxygen to attack the reinforcing steel, forming iron oxide (i.e., rust). Chloride ions are introduced into the concrete by marine spray, industrial brine, or deicing agents. These chloride ions can reach the reinforcing steel by diffusing through the concrete or by penetrating cracks in the concrete. An inspector may see rebar rust stains on the outer concrete surfaces before a spall occurs. The corrosion product (rust) can occupy up to 10 times the volume of the corroded steel that it replaces. This expansive action creates internal pressures up to 20.7 MPa (3000 psi) that will cause the concrete to yield, resulting in wider cracks, delaminations, and spalls (see Figure 2.2.25).

Figure 2.2.25 Corroded Reinforcing Bar

2.2.7 Prestressed Concrete Deterioration

Prestressed concrete members deteriorate in a similar fashion to ordinary concrete members. However, the effects on Prestressed concrete member performance are usually more detrimental. Significant defects include:

- Structural cracks
- Exposed prestressing tendons
- Corrosion of tendons in the bond zone
- Loss of camber due to concrete creep
- Loss of camber due to lost prestress forces

Structural cracks indicate an overload condition has occurred. These cracks expose the tendons to the environment, which can lead to corrosion.

Exposed steel tendons via cracks or collision damage corrode at an accelerated rate due to the high tensile stresses carried and can fail prior to any measurable section loss due to environmentally induced cracking (EIC).

Environmentally induced cracking in steel prestressing strands can occur when the steel prestressing strands are subject to high tensile stresses in a corrosive environment. Rust stains may be present. The strands, which are normally ductile, undergo a brittle failure due to the combination of the corrosive environment along with the tensile stresses.

There are two types of environmentally induced cracking. The first is called stress corrosion cracking (SCC). This type of cracking grows at a slow rate and has a branched cracking pattern. The corrosion of prestressing steel along with the tensile stress in the steel causes a cracking pattern perpendicular to the stress direction.

The second type is called hydrogen-induced cracking (HIC) and occurs due to hydrogen diffusing into the prestressing steel. Once in the steel, hydrogen gas is formed. The hydrogen gas applies an internal pressure to the prestressing steel. This internal pressure, in conjunction with the tensile stress due to prestressing, has the ability to create very brittle, non-branching, fast growing cracks in the prestressing steel strands. The specific type of environmentally induced cracking can only be positively identified after failure through the use of advanced inspection techniques.

When deteriorated concrete cover allows corrosion of the tendons in the bond zone (the end thirds of the beam), loss of development occurs which reduces prestress force. This can sometimes be evidenced by reduced positive camber and ultimately structural cracking. Prestress force can also be reduced through a beam-shortening phenomenon called creep, which relaxes the steel tendons. Loss of prestress force is followed by structural cracking at normal loads due to reduced live load capacity.

2.2.8 Other Causes of Concrete Deterioration

Temperature Changes

Freezing and thawing are common causes of concrete deterioration (see Figure 2.2.26). Porous concrete absorbs water, and when this water freezes, high expansive pressures are created due to the larger volume created by ice formation. These pressures often produce cracking and light spalling. Freeze/thaw damage should not be confused with scaling.

Figure 2.2.26 Freeze-Thaw Damage on a River Pier

Chemical Attack

Aside from accelerated rebar corrosion, the use of salt or chemical deicing agents contributes to weathering through recrystallization. This is quite similar to the effects of freezing and thawing.

Sulfate compounds in soil and water are also a problem. Sodium, magnesium, and calcium sulfates react with compounds in cement paste and cause rapid deterioration of the concrete.

Alkali-silica reaction (ASR) is a reaction between the alkalis in cement with the silica molecules of various aggregates. When the reaction takes place, a gel-like substance is formed. Once exposed to moisture, the gel expands and causes cracking in the concrete.

Moisture Absorption

All concrete is porous and will absorb water to some degree. As water is absorbed, the concrete will swell. If restrained, the material will burst or the concrete will crack. This type of deterioration is limited to concrete members that are continuously submerged in water.

Differential Foundation Movement

Foundation movement can also cause serious cracking in concrete structures. Differential settlement induces stresses in the supported superstructure and can lead to concrete deterioration.

Design and Construction Deficiencies

Some conditions or improper construction methods that can cause concrete to deteriorate are:

- Insufficient reinforcement bar cover - Insufficient concrete cover over rebars may lead to early corrosion of the steel reinforcement.

- Weep holes and scuppers - Improper placement or inadequate sizing of scuppers and weep holes can cause an accumulation of water with its damaging effects.

- Leaking deck joints
- Improper curing - A primary cause of concrete deterioration (loss of strength).
- Soft spots - Soft spots in the subgrade of an approach slab will cause the slab to settle and crack.
- Premature form removal - If the formwork is removed between the time the concrete begins to harden and the specified time for formwork removal, cracks will probably occur.
- Improper vibration - If the concrete is not properly vibrated, internal settling of the concrete mix can cause surface cracking above the reinforcing bars as the mix settles around the bars. Excessive vibration may cause segregation (separation of water, aggregate, and cement) of the concrete mix.
- Impurities - The inclusion of clay or soft shale particles in the concrete mix will cause small holes to appear in the surface of the concrete as these particles dissolve. These holes are known as mudballs.
- Internal voids - If reinforcing bars are too closely spaced, voids, which collect water, can occur under the reinforcing mat if the mix is not properly vibrated.

Fire Damage

- Extreme heat will damage concrete. High temperatures (above 370°C (700°F)) will cause a weakening in the cement paste and lead to cracking.

2.2.9 Protective Systems

Types and Characteristics of Concrete Coatings

Coatings form a protective barrier film on the surface of concrete to preclude entry of water and chlorides into the porous concrete. The practice of coating the concrete surface varies with each agency. Two primary concrete coatings are paint and water repellent membranes.

Paint

Paint is applied in one or two layers. The first layer fills the voids in a rough concrete surface. The second layer forms a protective film over the first. On smooth concrete surfaces, only one layer may be necessary.

Several classes of paint are coated on concrete:

- Oil-based paint
- Latex paint
- Epoxy paint

- Urethanes

Oil-based Paint

Oil-based paint is declining in use but is still found on some older concrete structures. Oil paint is subject to saponification failure in wet areas. Saponification is a chemical attack on the coating caused by the inherent alkalinity of the concrete. The moisture may be from humidity in the atmosphere, rain runoff, or ground water entering the porous concrete from below. Saponification does not occur over dry concrete (or occurs at a greatly reduced rate).

Latex Paint

Latex paint consists of a resin emulsion. Latexes can contain a variety of synthetic polymer binding agents. Latex paint resists attack by the alkaline concrete. Acrylic or vinyl latexes provide better overall performance, in that they are more resistant to alkaline attack than oil-based paint. Latex paints, however, are susceptible to efflorescence. Efflorescing is a process in which water-soluble salts pass outward through concrete and are deposited at the concrete/paint interface. This can cause loss of coating adhesion. If the paint is also permeable to water, the salts are deposited on the paint surface as the water evaporates.

Acrylics do not chalk as rapidly as other latexes and have good resistance to ultraviolet rays in sunlight. Polyvinyl acetate latexes are the most sensitive to attack by alkalis.

Epoxy Paint

Epoxy paint uses a cross-linking polymer binder, in which the epoxy resin in the paint undergoes a chemical reaction as the paint cures, forming a tough, cross-linked paint layer. Epoxies have excellent resistance to chemicals, water, and atmospheric moisture. Most epoxies are sensitive to the concrete's moisture content during painting. Polyamide-cured and water-base epoxy systems, however, have substantially overcome the moisture intolerance problem. For other epoxy systems, the concrete moisture should be measured prior to painting.

Urethanes

Urethanes are usually applied over an epoxy primer. They provide excellent adhesion, hardness, flexibility, and resistance to sunlight, water, harmful chemicals, and abrasion. They are, however, sensitive to temperature and humidity during application. The urethanes used on concrete require moisture to cure. In high humidity, the paint cures too quickly, leaving a bubbly appearance.

Many states now apply moisture-cured urethane anti-graffiti coatings on accessible concrete structures (see Figure 2.2.27). These are smooth, clear coatings applied without a primer coat. Spray paint and indelible marker ink adhere poorly to the smooth urethane, permitting easier cleaning than if they were applied to porous concrete.

Figure 2.2.27 Anti-Graffiti Coating on Lower Area of Bridge Piers

Water Repellent Membranes

Water repellent membranes (sealers) applied to concrete bridge decks, piers, abutments, columns, barriers, or aprons form a tight barrier to water and chlorides. The membrane penetrates up to 10 mm (3/8 inch) into the concrete to give strong adhesion. Membranes have good resistance to abrasion from weathering and traffic. Methyl methacrylate, silane, and silicone are three common water repellent coatings.

Surface Preparation

Concrete, as with any other surface, must be properly cleaned prior to coating. The surface may also require roughening to improve coating adhesion, as the forms used to mold concrete leave a surface that is too smooth for good coating adhesion. In addition, the oils applied to wooden forms to facilitate removal may impede coating adhesion.

Blast Cleaning

Blast cleaning with dry abrasives, high pressure water (up to 380 Mpa (55,000 psi)), or a water/abrasive mix is used to remove dirt, old paint, grease, and deteriorated concrete. It is also the best method to roughen the surface.

Open nozzle blast cleaning uses compressed air free of oil and moisture to propel the abrasive at speeds up to 645 kilometers per hour (400 miles per hour). Centrifugal wheel blast cleaning uses a rotating wheel to propel abrasive. The most common abrasive is sand, although many others, such as steel shot and grit, silica, aluminum oxide, and silicon carbide, are also available. Unlike dry abrasive blast cleaning, high pressure water can penetrate deep into concrete. The concrete must be allowed to dry thoroughly before a coating is applied.

Acid Etching

Acid etching is an efficient method of cleaning concrete. Hydrochloric acid (also called muriatic acid) reacts with the alkaline concrete surface, allowing surface contaminants to easily wash away. It leaves a roughened surface profile for good coating adhesion. All acid must be removed prior to coating application.

Types and Characteristics of Reinforcement Coatings

Because unprotected steel reinforcement corrodes and has adverse effects on concrete, some type of protective coating should typically be used on all steel reinforcement placed in concrete structures to ensure minimal steel corrosion. Steel reinforcement can be protected by the following methods:

- Epoxy coating
- Galvanizing
- Cathodic protection

Epoxy Coating

Epoxy coating is resistant to chemicals, water, and atmospheric moisture. Epoxies utilize an epoxy polymer binder, which forms a tough, resilient film upon drying and curing. Drying is by solvent evaporation, while curing entails a chemical reaction between the coating components. Epoxy coatings have excellent atmospheric exposure characteristics, as well as resistance to chemicals and water. They are often used as the intermediate coat in a three layer paint system. There are also two- and three-layer systems, which use only epoxies. One disadvantage of epoxies is that they chalk when exposed to sunlight. This chalking must be removed prior to topcoating with another layer of epoxy or another material. If not removed, the chalking will compromise subsequent adhesion.

Galvanizing

Another method of protecting steel reinforcement is by galvanizing the steel. This also slows down the corrosion process and lengthens the life of the reinforced concrete. This occurs by coating the bare steel reinforcement with zinc. The two unlike metals form an electrical current between them and one metal virtually stops its corrosion process while the other's accelerates due to the electrical current. In this situation, the steel stops corroding, while the zinc has accelerated corrosion.

Cathodic Protection

Steel reinforcement corrosion can also be slowed down by cathodic protection. Corrosion of steel reinforcing bars in concrete occurs by an electrical process in a moist environment at the steel surface. During corrosion, a voltage difference (less than 1 volt) develops between rebars or between different areas on the same rebar. Electrons from the iron in the rebar are repelled by the negative anode area of the rebar and attracted to the positive cathode area. This electron flow constitutes an electrical current, which is necessary for the corrosion process.

Corrosion occurs only at the anode, where the electrons from the iron are given up.

By cathodic protection, this electrical current is reversed, which slows or stops corrosion. By the impressed current method, an electrical DC rectifier supplies electrical current from local electrical power lines to a separate anode embedded in the concrete. The anode is usually a wire mesh embedded just under the concrete surface. Another type of anode consists of an electrically conductive coating applied to the concrete surface. The wires from the rectifier are embedded in the coating at regular intervals.

When the impressed current enters the mesh or coating anode, the voltage on the rebars is reversed, turning the entire rebar network into a giant cathode. Since natural corrosion occurs only at the anode, the rebars are protected.

The natural corrosion process is allowed to proceed with electrons leaving the iron atoms in the anode. With impressed current cathodic protection, however, the electrons are supplied from an external source, the DC rectifier. Thus, the artificial anode mesh or coating is also spared from corrosion.

During the bridge inspection, check that all visible electrical connections and wiring from the rectifier to the concrete structure are intact.

2.2.10 Inspection Procedures for Concrete and Protective Coatings

There are three basic procedures used to inspect prestressed and reinforced concrete members. Depending on the type of inspection, the inspector may be required to use only one individual procedure or all procedures. They include:

- Visual
- Physical
- Advanced inspection techniques

Visual Examination

There are two types of visual inspections that may be required of an inspector. The first, called a cursory inspection, involves reviewing the previous inspection report and visually examining the members from beneath the bridge. All concrete surfaces should receive a thorough visual assessment to identify obvious defects during a cursory inspection.

The second type of visual inspection is called a “hands-on” inspection. This type of visual inspection requires the inspector to visually assess all defective concrete surfaces at a distance no further than an arm’s length. The concrete surfaces are given close visual attention to quantify and qualify any defects.

Physical Examination

Areas of concrete or rebar deterioration identified visually should also be examined physically using an inspection hammer. This hands-on effort verifies the extent of the defect and its severity.

High stress areas should be sounded for defects using an inspection hammer. Hammer sounding is commonly used to detect areas of delamination and unsound concrete. For large horizontal surfaces such as bridge decks, a chain drag may be used. A chain drag is made of several sections of chain attached to a handle. The inspector drags this across a deck and makes note of the resonating sounds. A delaminated area will have a distinctive hollow “clacking” sound when tapped with a hammer or revealed with a chain drag. A hammer hitting sound concrete will result in a solid "pinging" type sound (see Figure 2.2.28).

The location and width of cracks found during the visual inspection and sounding procedures should be given special attention. For typical reinforced concrete members, a crack comparitor card can be used to measure the width of cracks. This type of crack width measuring device is a transparent card about the size of an identification card. The card has black lines on it that represent crack widths. The line on the card that best matches the width of the crack lets the inspector know the measured width of the crack. For prestressed members, crack widths are usually narrower in width. For this reason, a crack gauge, which is a more accurate crack width-measuring device, should be used. For crack width guidelines, see Table 2.2.3.

Figure 2.2.28: Inspector Using a Chain Drag

Advanced Inspection Techniques

If the extent of the concrete defect cannot be determined by the visual and/or physical inspection procedures described above, advanced inspection techniques should be used. Nondestructive methods, described in Topic 13.2.2, include:

- Acoustic wave sonic/ultrasonic velocity measurements
- Delamination detection machinery
- Electrical methods
- Electromagnetic methods
- Pulse velocity
- Flat jack testing
- Ground-penetrating radar
- Impact-echo testing
- Infrared thermography
- Laser ultrasonic testing
- Magnetic field disturbance
- Neutron probe for detection of chlorides
- Nuclear methods
- Pachometer
- Rebound and penetration methods
- Ultrasonic testing

Other methods, described in Topic 13.2.3, include:

- Core sampling
- Carbonation
- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes
- Moisture content
- Reinforcing steel strength

Destructive methods for protective coatings include:

- Probing
- Paint dry film thickness (Tooke Gauge)

Physical Examination of Protective Coatings

Areas to Inspect

While inspecting protective coatings, pay close attention to the following areas:

- Areas open to direct weathering by wind, rain, hail, or seawater spray.
- Roadway splash zones along curbs, parapets, and expansion dams. These areas are subject to impact abrasion by debris from passing vehicles.
- Inaccessible or hard-to-reach areas where coatings may be missing or improperly applied.
- All concrete joints.
- Areas that retain moisture or salt. Horizontal surfaces of concrete beams and piers are common examples. Also inspect areas where drainage systems deposit salt and water, such as beneath catch basins, scuppers, downspouts, and bearing areas.
- Impact areas on bridge decks and parapets where snowplows or vehicle accidents damage coatings.

Coating Failures

The following failures are characteristic of paint on concrete:

- Lack of adhesion/peeling can be caused by poor adhesion of the primer layer to the concrete or by poor bonding between coating layers. Waterborne salts depositing under a water-impermeable coating (efflorescence) will also cause a coating to peel.
- Chalking is a powdery residue left on paint as ultraviolet light degrades the paint.
- Erosion is a gradual wearing away of a coating. It is caused by abrasion from wind-blown sand, soil and debris, rain, hail, or debris propelled by motor vehicles.
- Checking is composed of short, irregular breaks in the top layer of paint, exposing the undercoat.
- Cracking is similar to checking, but with cracking, the breaks extend completely through all layers of paint to the concrete substrate.
- Microorganism failure occurs as bacteria and fungi feed on paint containing biodegradable components. The damp nature of concrete makes it susceptible to this type of paint failure.
- Saponification results from a chemical reaction between concrete, which is alkaline, and oil-based paint. It destroys the paint, leaving a soft residue.
- Wrinkling is a rough, crinkled paint surface due to excessive paint thickness or high temperature during painting. It is caused by the surface of the paint film at the air interface solidifying before solvents have had a chance to escape from the interior of the paint film.

Table of Contents

Section 2
Bridge Materials

Steel Description	Steel Designation: American Society for Testing and Materials (ASTM)	Steel Designation: American Association of State Highway and Transportation Officials (AASHTO)	Years in Use
Structural Carbon Steel	A7	M94	1900-1967
Structural Nickel Steel	A8	M96	1912-1962
Structural Steel	A36 (A709 Grade 36)	M183 (M270 Grade 36)	1960-Present (1974-Present)
Structural Silicon Steel	A94	M95	1925-1965
Structural Steel	A140		1932-1933
Structural Rivet Steel	A141	M97	1932-1966
High-Strength Structural Rivet Steel	A195	M98	1936-1966
High-Strength Low-Alloy Structural Steel	A242	M161	1941-Present
Low and Intermediate Tensile Strength Carbon Steel Plates	A283		1946-Present
Low and Intermediate Tensile Strength Carbon-Silicon Steel Plates	A284		1946-Present
Steel Sheet Piling	A328	M202	1950-Present
Structural Steel for Welding	A373	M165	1954-1965
High-Strength Structural Steel	A440	M187	1959-1979
High-Strength Low-Alloy Structural Manganese Vanadium Steel	A441	M188	1954-1989
High-Yield-Strength, Quenched and Tempered Alloy Steel Plate (Suitable for Welding)	A514 (A709 Grade 100/100W)	M244 (M270 Grade 100/100W)	1964-Present (1974-Present)
Hi Strength Low-Alloy Columbium-Vanadium Steel of Structural Quality	A572 (A709 Grade 50)	M223 (M270 Grade 50)	1966-Present (1974-Present)
Hi-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 inches Thick	A588 (A709 Grade 50W)	M222 (M270 Grade 50W)	1968-Present (1974-Present)
High-Strength Low-Alloy Steel H-Piles and Sheet Piling	A690		1974-Present
Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi Minimum Yield Strength to 4 inches Thick	A852 (A709 Grade 70W)	M313 (M270 Grade 70W)	1985-Present (1985-Present)

Summary of Steel Designations (Primary Source: Beer and Johnston, *Mechanics of Materials*, New York: McGraw-Hill, 1981)

Topic 2.3 Steel

2.3.1 Introduction

Steel is a widely used construction material for bridges due to its strength, relative ductility, and reliability. It is found in a variety of members on a large number of bridges. Therefore, the bridge inspector should be familiar with the various properties and types of steel.

2.3.2 Common Methods of Steel Member Fabrication

Rolled Beams

Rolled beams are manufactured in structural rolling mills. The flanges and web are one piece of steel. Rolled beams in the past were generally available no deeper than 914 mm (36”) in depth but are now available from some mills as deep as 1120 mm (44”).

Rolled beams are generally compact’ sections which satisfy flange to web thickness ratios to prevent buckling.

Rolled beams generally will have bearing stiffeners but no intermediate stiffeners since they are compact.

Plate Girders

These are larger members than can be provided for in a rolling mill. In other words, larger than 914 mm (36”) or 1120 mm (44”).

These are built-up shapes which are composed of any combination of plates, bars, and rolled shapes. The term “built-up” describes the way the final shape is made.

Older fabricated multi-girders were constructed of riveted built-up members. Today’s fabricated multi-girders are constructed from welded members.

2.3.3 Common Steel Shapes Used in Bridge Construction

Steel as a bridge construction material is available as wire, cable, plates, bars, rolled shapes, and built-up shapes. Typical areas of application for the various types of steel shapes are listed below:

- Wires are typically used as prestressing strands or tendons in beams and girders (See Figure 2.2.4).
- Cable-stay and steel suspension bridges are primarily supported by steel cables (see Figure 2.3.1).
- Steel plates have a wide variety of uses. They are primarily used to construct built-up shapes (see Figure 2.3.2).
- Steel bars are generally placed in concrete to provide tensile reinforcement in the form of deformed round bars. Steel bars can also be used as tension bracing members or with steel plates to construct grid systems (see Figure

2.3.3).

- Rolled shapes are used as structural beams and columns and are made by placing a block of steel through a series of rollers that transform the steel into the desired shape. These steel shapes are either hot rolled or cold rolled. The typical rolled shape is an "I" shape. The "I" shape comes in many sizes and weights (see Figure 2.3.4). They can also be fabricated with a straight or tapered flange thickness. Other rolled shapes are channel or "C" shapes, angles, and "T" shapes.

- Built-up shapes are also used as structural beams and columns but are composed of any combination of plates, bars, and rolled shapes. The term "built-up" describes the way the final shape is made. Built-up shapes are used when an individual rolled shape cannot carry the required load or when a unique shape is desired. Built-up shapes are riveted, bolted, or welded together. Common built-up shapes include I-girders, box girders, and truss members (see Figure 2.3.5).

Figure 2.3.1 Steel Cables

Figure 2.3.2 Steel Plates

Figure 2.3.3 Steel Bars

Figure 2.3.4 Rolled Shapes

Figure 2.3.5 Built-up Shapes

2.3.4
Properties of Steel

Physical Properties

Many of the nation's largest bridges are constructed primarily of steel. When compared with iron, steel has greater strength characteristics, it is more elastic, and it can withstand the effects of impact and vibration better.

Although iron has some carbon chemically dissolved in it, when the carbon content is greater than 0.l%, the material is classified as steel. Steel has a unit weight of about 7850 kg/m^3 (490 pcf).

ASTM and AASHTO define the required properties for various steel types. ASTM classifies each type with an "A" designation, while AASHTO uses an “M” designation.

Low carbon steel, steel with carbon content less than approximately 0.3%, defines some of the most common steel types:

- A7 steel - the most widely used bridge steel up to about 1967; obsolete due to poor weldability characteristics
- A373 steel - similar to A7 steel but has improved weldability characteristics due to controlled carbon content
- A36 steel - the latest of the low carbon steels (first used in 1960); it features good weldability and improved strength

Structural nickel steel (A8) was used widely prior to the 1960's in bridge construction, but welding problems occurred due to relatively high carbon content.

Structural silicon steel (A94) was used extensively in riveted or bolted bridge structures prior to the development of low alloy steels in the 1950's. This steel also has poor weldability characteristics due to high carbon content.

Quenched and tempered alloy steel plate (A514) was developed primarily for use in welded bridge members.

High strength, low alloy steel is used where weight reduction is required, where increased durability is important, and where atmospheric corrosion resistance is desired; examples include:

- A441 steel - manganese vanadium steel
- A572 steel - columbium-vanadium steel (replaced by A441 in 1989)
- A588 steel - a "weathering steel," was developed to be left unpainted, which develops a protective oxide coating upon exposure to the atmosphere under proper design and service conditions (refer to Topic 2.3.5 for a further description of weathering steel)

These steels are also copper bearing, which provides increased resistance to atmospheric corrosion and a slight increase in strength.

Some of the steel types listed above were used widely in the past but are no longer being manufactured. A new ASTM designation (A709) was developed in 1974. This designation covers carbon and high-strength low-alloy steel structural shapes, plates, and bars, and quenched and tempered alloy steel for structural plates intended for use in bridges. Six grades are available in four yield strength levels (36, 50, 70, and 100). The steel grade is equivalent to the yield strength in units of kips per square inch (ksi). Grades 36, 50, 50W, 70W, and 100/100W are also included in ASTM Specifications A36, A572, A588, A852, and A514, respectively. Grades 50W, 70W, and 100W have enhanced atmospheric corrosion resistance and are labeled with a "W" for weathering steel.

In 1996, a new steel type, High Performance Steel (HPS), was introduced to bridge construction. This type of steel was designed to improve weldability, toughness, and atmospheric corrosion resistance. Prior to the new steel designs, a set of "goal properties" was implemented and then testing took place to meet the goals. The first grade of HPS was HPS-70W, which was produced by Thermo-Mechanical-Controlled Processing (TMCP). The HPS-70W has improved Charpy V-Notch impact properties compared to 70W. Currently the HPS grades available are HPS-50W, HPS-70W, and HPS-100W.

In addition to the ASTM steel designations, the American Association of State Highway and Transportation Officials (AASHTO) also publishes its own steel designation (M270). For each ASTM steel designation, there is generally a corresponding AASHTO steel designation. For a summary of the various ASTM and AASHTO steel designations, refer to the table at the beginning of Topic 2.3.

Mechanical Properties

Some of the mechanical properties of steel include:

- Strength - steel is isotropic and possesses great compressive and tensile strength, which varies widely with type of steel
- Elasticity - the modulus of elasticity is nearly independent of steel type and is commonly assigned as 200,000 MPa (29,000,000 psi)
- Ductility - both the low carbon and low alloy steels normally used in bridge construction are quite ductile; however, brittleness may occur because of heat treatment, welding, or metal fatigue
- Fire resistance - steel is subject to a loss of strength when exposed to high temperatures such as those resulting from fire (see Topic 2.3.4 – for specific temperature information)
- Corrosion resistance - unprotected carbon steel corrodes (i.e., rusts) readily; however, steel can be protected
- Weldability - steel is weldable, but it is necessary to select a suitable welding procedure based on the chemistry of the steel
- Fatigue - fatigue problems in steel members and connections can occur in bridges due to numerous live load stress cycles combined with poor weld or connection details

2.3.5 Types and Causes of Steel Deterioration

Corrosion

To properly inspect a steel bridge, the inspector must be able to recognize the various types of steel defects and deterioration. The inspector must also understand the causes of the defects and how to examine them. The most recognizable type of steel deterioration is corrosion (see Figure 2.3.6). Bridge inspectors should be familiar with corrosion since it can lead to a substantial reduction in member capacity. Corrosion is the primary cause of section loss in steel members and is most commonly caused by the wet-dry cycles of exposed steel. When deicing chemicals are present, the effect of moisture is accelerated.

Figure 2.3.6 Steel Corrosion and Complete Section Loss on a Beam Web

Some of the common types of corrosion include:

- Environmental corrosion - primarily affects metal in contact with soil or water and is caused by formation of a corrosion cell due to deicing salt concentrations, moisture content, oxygen content, and accumulated foreign matter such as roadway debris and bird droppings
- Stray current corrosion - caused by electric railways, railway signal systems, cathodic protection systems for pipelines or foundation pilings, DC industrial generators, DC welding equipment, central power stations, and large substations
- Bacteriological corrosion - organisms found in swamps, bogs, heavy clay, stagnant waters, and contaminated waters can contribute to corrosion of metals
- Stress corrosion - occurs when tensile forces expose an increased portion of the metal at the grain boundaries, leading to corrosion and ultimately fracture
- Fretting corrosion - takes place on closely fitted parts which are under vibration, such as machinery and metal fittings, and can be identified by pitting and a red deposit of iron oxide at the interface

Fatigue Cracking

Another type of deterioration is fatigue cracking (see Figure 2.3.7). Fatigue failure occurs at a stress level below the yield stress and is due to repeated loading. Fatigue cracking has occurred in several types of bridge structures around the nation. This type of cracking can lead to sudden and catastrophic failure on certain bridge types. Therefore, the bridge inspector should know where to look and how to recognize early stages of fatigue crack development.

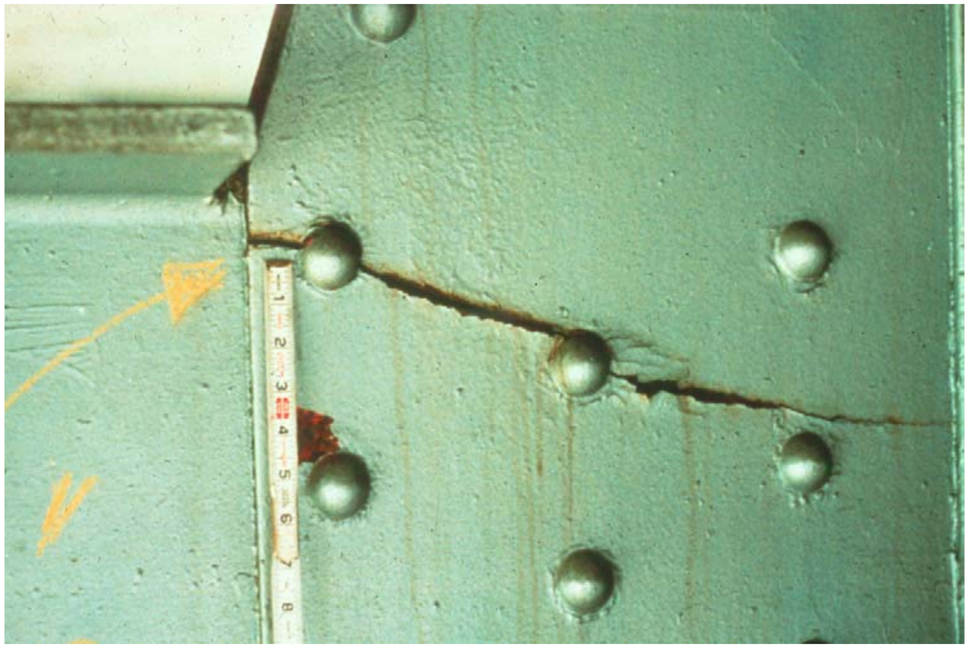


Figure 2.3.7 Fatigue Crack at Coped Top Flange of Riveted Connection

Some factors leading to the development of fatigue cracks are:

- Frequency of truck traffic
- Age or load history of the bridge
- Magnitude of stress range
- Type of detail
- Quality of the fabricated detail
- Material fracture toughness (base metal and weld metal)
- Weld quality
- Ambient temperature

There are two basic types of bending in bridge members: in-plane and out-of-plane. When in-plane bending occurs, the cross section of the member resists the load according to the design and undergoes nominal elastic deformation. Out-of-plane bending implies that the cross section of the member is loaded in a plane other than that for which it was designed and undergoes significant elastic deformation or distortion. More correctly, out-of-plane bending should be referred to as out-of-plane distortion. Out-of-plane distortion is common in beam webs where transverse members connect and can lead to fatigue cracking (see Figure 2.3.8).

There is a distinction between fatigue that is caused from in-plane (as designed) bending and out-of-plane distortion.

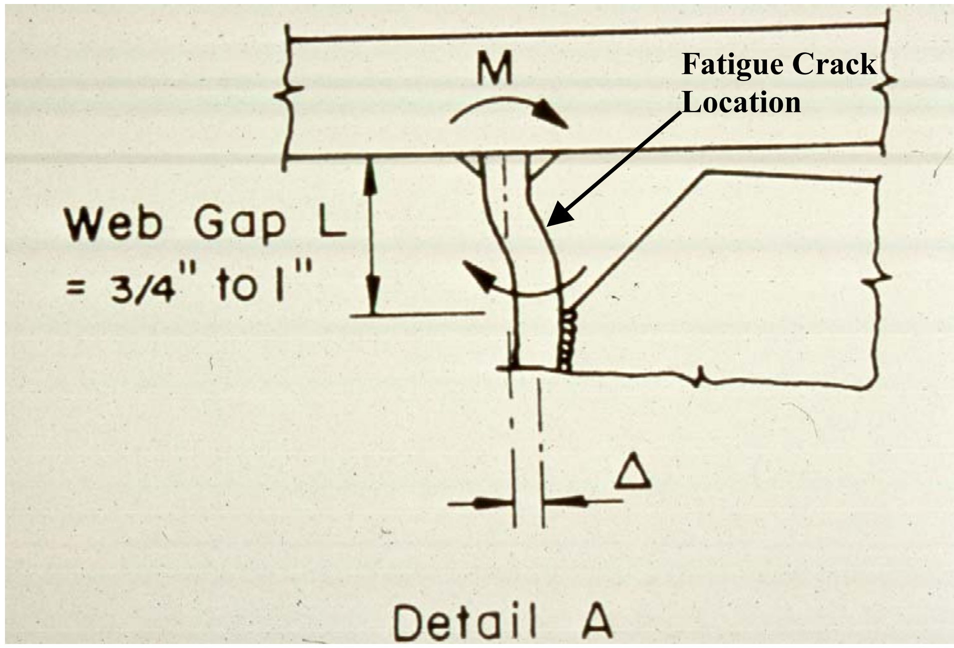


Figure 2.3.8 Out-of-plane Distortion

Additional information about fatigue and fracture in steel bridges is presented in Topic 8.1.

Overloads

Overloads are loads that exceed member or structure design loads. Steel is elastic (i.e., it returns to the original shape when a load is removed) up to a certain point, known as the yield point (see Topic P.1). When yield occurs, steel will bend or elongate and remain bent or elongated after the load has been removed. This type of permanent deformation of material beyond the elastic range is called plastic deformation. Plastic deformations due to overload conditions may be encountered in both tension and compression members.

The symptoms of plastic deformation in tension members are:

- Elongation
- Decrease in cross section, commonly called "necking down"

The symptoms of plastic deformation in compression members are:

- Buckling in the form of a single bow
- Buckling in the form of a double bow or "S" type, usually occurring where the section under compression is pinned or braced at the center point

An overload can lead to plastic deformation, as well as complete failure of the member and structure. This occurs when a tension member breaks or when a compression member exhibits buckling distortion at the point of failure.

Collision Damage

Components and structural members of a bridge that is adjacent to a roadway or waterway are susceptible to impact damage. Indications of impact damage include dislocated and distorted members (see Figure 2.3.9).

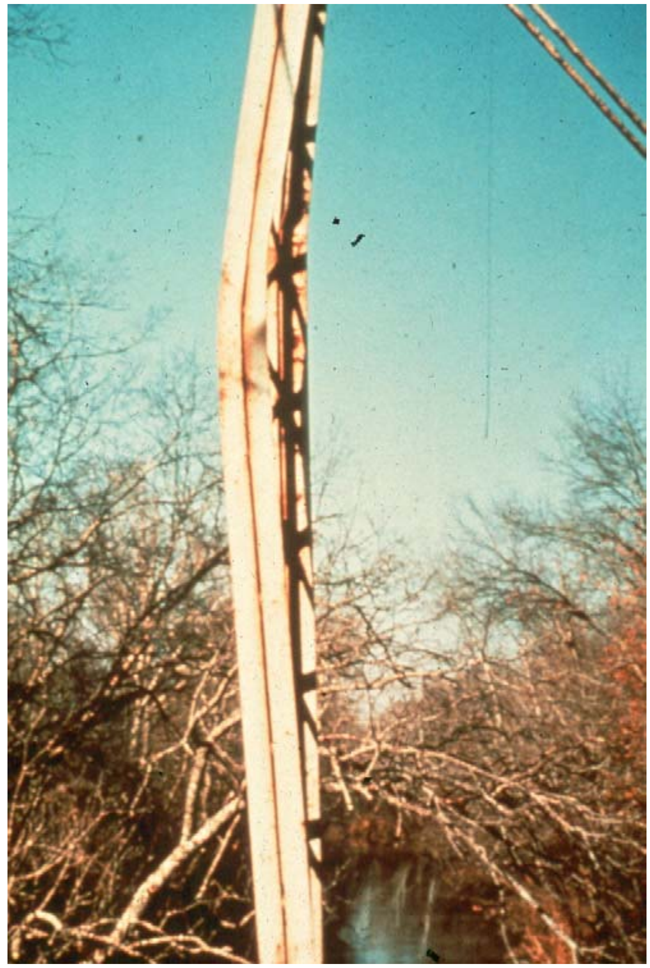


Figure 2.3.9 Collision Damage on a Steel Bridge

Heat Damage

Steel members will undergo serious deformation upon exposure to extreme heat (see Figure 2.3.10). In addition to sagging, or elongation of the metal, intense heat often causes members to buckle and twist; rivets and bolts may fail at connection points. Buckling could be expected where the member is under compression, particularly in thin sections such as the web of a girder.

Figure 2.3.10 Heat Damage

Temperatures affecting steel strength are as follows:

- 204–260°C (400°-500°F) - starts to affect strength
- 482–538°C (900°-1000°F) - major loss of strength

Paint Failures

The following paint failures are common on steel:

- Chalking, erosion, checking, cracking, and wrinkling (see Figure 2.3.11), as described in ASTM D-3359.
- Blisters are caused by painting over oil, grease, water, salt, or by solvent retention. Corrosion can occur under blisters.
- Undercutting occurs when surface rust advances under paint. It commonly occurs along scratches that expose the steel or along sharp edges (see Figure 2.3.12). The corrosion undermines intact paint, causing it to blister and peel.
- Pinpoint rusting can occur at pinholes in the paint, which are tiny, deep holes in the paint, exposing the steel (see Figure 2.3.13). It can also be caused by thin paint coverage. In this case, the "peaks" of the roughened steel surface protrude through the paint and corrode.
- Microorganism failure is caused by bacteria or fungi attacking biodegradable coatings. Oil/alkyds are the most often affected.
- Alligatoring can be considered a widely spaced checking failure, caused by internal stresses set up within the surface of a coating during drying (see Figure 2.3.14). The stresses cause the surface of the coating to shrink more rapidly to a much greater extent than the body of the coating. This causes large surface checks that do not reach the steel substrate.

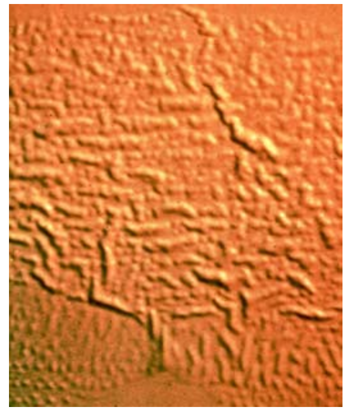


Figure 2.3.11 Paint Wrinkling

Figure 2.3.12 Rust Undercutting at Scratched Area

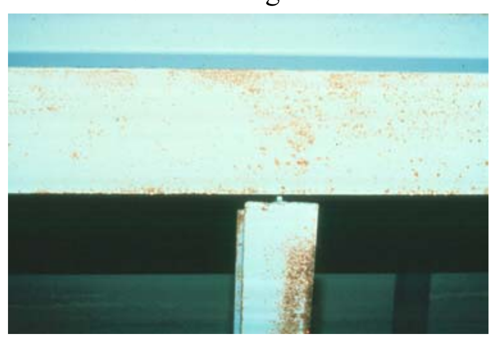


Figure 2.3.13 Pinpoint Rusting

Figure 2.3.14 Paint Peeling from Steel Bridge Members

- Mudcracking can be considered a widely spaced cracking failure, where the breaks in the coating extend to the steel substrate, allowing rapid corrosion (see Figure 2.3.15). Mudcracking is often a phenomenon of inorganic zinc-rich primers, which are applied as a very thick layer or are applied on a hot surface. Rapid curing causes the shrinkage, which yields the alligatoring, and ultimately, mudcracks.
- Bleeding occurs when soluble colored pigment from an undercoat penetrates the topcoat, causing discoloration.

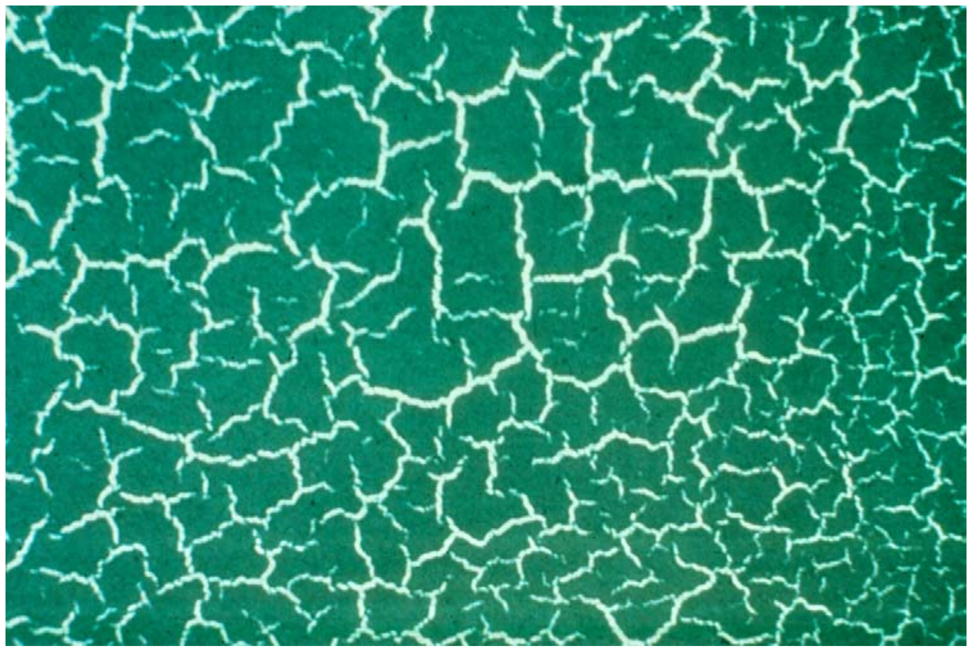


Figure 2.3.15 Mudcracking Paint

2.3.6 Factors that Influence Fatigue Life

Main factors influencing the development of fatigue cracks are:

1. **Magnitude of Stress Range**
   Trucks (not cars) produce stress ranges that may lead to fatigue cracks.

2. **Number of Cycles (Frequency)**
   This is dependent on:
   - Frequency of truck traffic
   - Age or load history of the bridge

3. **Fatigue Prone Details**
   This depends on:
   - The type of detail
   - Quality of the fabricated detail
   - Weld quality

Other factors influencing the development of fatigue cracks are:

Material Fracture Toughness
Toughness of the:
- Base metal
- Weld metal

Toughness is based on the chemical composition of the steel.

Ambient Temperature
- Colder – more likely to crack

Fatigue and fracture will be discussed in further detail in Topic 8.1 and the "Fracture Critical Inspection Techniques for Steel Bridges" NHI Course number 130078.

2.3.7 Protective Systems

Protective systems, when applied properly, provide protection needed against rust or corrosion**.**

Corrosion of Steel

Painting and weathering are the primary means used to protect structural steel from rust and corrosion. To understand how paint or formation of protective rust prevents corrosion and how to inspect paint coatings, it is first necessary to understand the corrosion process.

Corrosion can be defined as a wearing away of metal by a chemical or electrochemical oxidizing process. Corrosion in metals is a form of oxidation caused by a flow of electricity from one part of the surface of one piece of metal to another part of the same piece. The result is the conversion of metallic iron to iron oxide. Once the corrosion process takes place, the steel member has a loss of section which results in a loss of structural capacity. Both conduction and soluble oxygen are necessary for the corrosion process to occur.

A conductive solution (water) or electrolyte must be present in order for current to flow. Corrosion occurs very slowly in distilled water, but much faster in salty water, because the presence of salt (notably sodium chloride) improves the ability of water to conduct electricity and contributes to the corrosion process. In the absence of chlorides, steel (iron) corrodes slowly in the presence of water. Water is both the medium in which corrosion normally occurs and provides the corrosion reaction. In addition, oxygen accelerates the corrosion process. Corrosion stops or proceeds at a reduced rate when access to water and oxygen is eliminated or limited. Water and oxygen are therefore essential for the corrosion process. For example, corrosion of steel does not occur in moisture-free air and is negligible when the relative humidity of the air is below 30% at normal or lower temperatures. The presence of chlorides in the water will accelerate corrosion by increasing the conductivity of the water.

To have corrosion take place in steel, then, one must have:

- Oxygen
- An electrolyte to conduct current
- An area or region on a metallic surface with a negative charge (cathode)

- An area or region on the metallic surface with a positive charge (anode)

Exposure of steel to the atmosphere provides a plentiful supply of oxygen. The presence of oxygen can limit corrosion by the formation of corrosion product films that coat the surface and prevent water and oxygen from reaching the uncorroded steel. The presence of contaminants such as chlorides accelerates the corrosion rate on steel surfaces by disrupting the protective oxide film.

Galvanic Action

The term "galvanic action" is generally restricted to the changes in normal corrosion behavior that result from the current generated when one metal is in contact with a different one. The two metals are in a corrosive solution when one metal may become an anode when it contacts a dissimilar metal. In such a "galvanic couple," the corrosion of one of the metals (e.g., zinc) will be accelerated, and the corrosion of the other (e.g., steel) will be reduced or possibly stopped. Galvanized coatings on highway guardrails and zinc-rich paint on structural steel are examples of galvanic protection using such a sacrificial (zinc) anode.

Types and Characteristics of Steel Coatings

Surface Preparation for Painting

The steel surface must be properly prepared prior to paint application. All foreign material must be removed. The following steps must be taken when preparing the surface for painting:

- Dirt and dust particles or spent abrasive from blast cleaning interfere with paint adhesion to the steel substrate and prevent application of a smooth, uniform film of paint. Debris embedded in the paint can also wick moisture and corrosive elements through the film to the substrate.
- Rust cannot be penetrated by most paints. Rust can become poorly adherent. In such cases, disbonding of the rust carries away the paint layers, permitting accelerated corrosion.
- Flash rust is a light layer of rust, which forms on the cleaned steel soon after exposure to the air, particularly in moist or humid environments. This layer may not be thoroughly wetted and may impede adhesion.
- Salts trapped in the paint film can cause blistering and disbonding.
- Oil and grease prevent good paint adhesion and must be completely removed. Welding smoke and inspection markings leave an oily residue that must also be completely removed.
- Dead paint that is loose, cracking, or flaking will eventually lift from the surface, carrying any new paint with it.
- Mill scale is a layer of iron oxide on the surface of steel. It forms when the steel is heated at high temperatures in a furnace. Mill scale has a bluish, somewhat shiny appearance, which may be difficult to see on partially blastcleaned steel. It must be completely removed when using most coating materials, as it may disbond upon expansion and contraction, carrying the paint with it.
- Weld spatter may also dislodge, leaving a bare exposed steel surface.

The surface should also be roughened to promote paint adhesion, as paint will not adhere well to a smooth surface.

Methods of Surface Preparation

The Society of Protective Coatings publishes a set of standards and specifications describing the following methods of surface preparation:

- Solvent cleaning
- Hand tool cleaning
- Power tool cleaning
- Abrasive blast cleaning
- Water blast cleaning

Solvent cleaning removes oil and grease. It is usually used in conjunction with or prior to the mechanical preparation methods. Common solvents include petroleum and coal tar solvents, turpentine, mineral spirits, alkaline cleaners, and emulsion cleaners, which contain oil soaps mixed with kerosene or mineral spirits.

Hand tool cleaning is used for removing loosely adhering paint, rust, or mill scale. It will not remove tightly adhering mill scale, or dirt and oils in crevices. Due to its slow speed, hand tool cleaning is used mostly for small area spot cleaning. Common hand tools include scrapers, wire brushes, chipping hammers, knives, chisels, and abrasive pads.

Power tool cleaning is effective on both plane and contoured steel surfaces. Power tool cleaning devices remove loose paint, rust, and scale. Power tools do not leave the residue common with blast cleaning. Also, power tools are used on small areas and where the abrasive could damage sensitive surroundings.

Abrasive blast cleaning is the preferred surface preparation method for coatings, which require a high degree of cleanliness and a uniformly roughened surface profile. Blast cleaning is a high production method, which can remove mill scale. A water collar is sometimes used with abrasive blast cleaning to prevent abrasive rust and paint particles from becoming airborne.

Water blast cleaning (hydroblasting) may be high or low pressure, hot or cold, with or without detergent, depending upon the type of cleaning desired. Water does not etch a steel surface and may not remove tight paint, rust, or mill scale. Abrasives may be injected into the water stream to remove tightly adhering material for faster clearing or to produce a roughened surface profile. Sand is the most common abrasive. The process can remove all old paint, rust, and mill scale. It yields a degree of cleanliness equivalent to open nozzle abrasive blast cleaning. Due to flash rusting caused by the high pressure water, water blast cleaned areas must be either cleaned by dry abrasive blast cleaning or a corrosion inhibiting chemical must be added to the high pressure water to prevent flash rusting.

Paint

Once the steel surface is properly prepared, the appropriate type and application of paint must be chosen based on the paint characteristics.

Paint is by far the most common coating used to protect steel bridges. Paint is composed of four basic compounds: pigments, vehicle (also called binder), solvents (also called thinners), and additives (such as thickeners and mildewoides). The pigments contribute such properties as inhibition of corrosion of the metal surface (e.g., zinc, zinc oxide, red lead, and zinc chromate), reinforcement of the dry paint film, stabilization against deterioration by sunlight, color, and hardness. Pigments are generally powders before being mixed into paint. The vehicle also remains in the dry-cured paint layer. It binds the pigment particles together and provides adhesion to the steel substrate and to other paint layers. Thus, the strength of the binder contributes to the useful life of the coating. Paint can be classified as inorganic or organic, depending on the vehicle. Inorganic paint uses a water soluble silicate binder which reacts with water during paint curing. Most types of paint contain one of a variety of available organic binders. The organic binders cure (harden) by one or more of the following mechanisms:

- Evaporation of solvents
- Reaction with oxygen in the air
- Polymerization through the action of heat or a catalyst
- Combination of reactive components in the binder

Solvents, which are liquids (such as water and mineral spirits), are included in paint to transport the pigment-binder combination to the substrate, to lower paint viscosity for easier application, to help the coating penetrate the surface, and to wet the substrate. Since the solvent is volatile, it eventually evaporates from the dry paint film. Additives are special purpose ingredients that give the product extra performance features. For example, mildewoides reduce mildew problems, and thickeners lengthen the drying time for application in hot weather.

Paint used on steel bridges acts as a physical barrier to moisture, oxygen, and chlorides, all of which promote corrosion. While water and oxygen are important to corrosion, chlorides from deicing road salts or seawater spray accelerate the corrosion process significantly.

Paint Layers

Paint on steel is usually applied in up to three layers, or coats:

- Primer coat
- Intermediate coat
- Topcoat

The primer coat is in direct contact with the steel substrate. It is formulated to have good wetting and bonding properties and may or may not contain passivating (corrosion-inhibiting) pigments.

The intermediate coat must strongly adhere to the primer. It provides increased thickness of the total coating system, abrasion and impact resistance, and a barrier to chemical attack.

The topcoat (also called the finish coat) is typically a tough, resilient layer, providing a seal to environmental attack, water, impact, and abrasion. It is also formulated for an aesthetic appearance.

Types of Paint

A wide variety of paints are applied to steel bridges. All of them except some zinc-rich primers use an organic binder.

Oil/alkyd Paint

Oil/alkyd paints use an oil such as linseed oil and an alkyd resin as the binding agent. Alkyd resin is synthetically produced by reacting a drying oil acid with an alcohol. Alkyd paints are low cost, with good durability, flexibility, and gloss retention. They are also tough, with moderate heat and solvent resistance. They should not be used in water immersion service or in alkaline environments.

A disadvantage is their offensive odor. They are also slow drying, difficult to clean up, and have poor exterior exposure. Alkyd paints often contain lead pigments, which are known to cause numerous health problems. The removal and disposal of lead-based paints is a regulated activity in all states.

Vinyl Paint

Vinyl paints are based on various vinyl polymer binding agents dissolved in a strong solvent. These paints cure by solvent evaporation. Vinyls have excellent chemical, water, salt, acid, and alkali resistance, good gloss retention, and are applicable at low temperatures. Conversely, their disadvantages include poor heat and solvent resistance, and poor adhesion. Vinyls are usually not used with other types of paint in a paint system. Vinyl coatings can be formulated to serve as primer, intermediate, and topcoat in paint systems.

Epoxies

Epoxies utilize an epoxy polymer binder, which forms a tough, resilient film upon drying and curing. Drying is by solvent evaporation, while curing entails a chemical reaction between the coating components. Epoxy coatings have excellent atmospheric exposure characteristics, as well as resistance to chemicals and water. They are often used as the intermediate coat in a three-layer paint system. There are also two- and three-layer systems, which use only epoxies. One disadvantage of epoxies is that they chalk when exposed to sunlight. This chalking must be removed prior to topcoating with another layer of epoxy or another material. If not removed, the chalking will compromise subsequent adhesion.

Epoxy Mastics

Epoxy mastics are heavy, high solid content epoxy paints, often formulated with flaking aluminum pigment. The mastics are useful in applications where a heavy paint layer is required in one application. They can be formulated with wetting and penetrating agents, which permit application on minimally prepared steel surfaces.

Urethanes

Urethanes are commonly used as the topcoat layer. They provide excellent sunlight resistance, hardness, flexibility (i.e., resistance to cracking), gloss retention, and resistance to water, harmful chemicals, and abrasion. All-urethane systems are also available which utilize urethane paints as primer, intermediate, and topcoat.

Zinc-rich Primers

Zinc-rich primers contain finely divided zinc powder (75% to 95%) and either an organic or inorganic binder. They protect the steel substrate by galvanic action, wherein the metallic zinc corrodes in preference to the steel. The materials have excellent adhesion and resist rust undercutting when applied over a properly prepared surface. The zinc-rich primers must be well mixed prior to application, or some coated areas will be deficient in zinc, lowering the substrate protection.

Latex Paint

Latex paint consists of a resin emulsion. The term covers a wide range of materials, each formulated for a different application. Latex on steel has excellent flexibility (allowing it to expand and contract with the steel as the temperature changes) and color retention, with good adhesion, hardness, and resistance to chemicals. Latex paint has low odor, faster drying time, and easier clean up.

The disadvantages of latex paint include sacrificed durability, and it must be applied at temperatures over 10°C (50°F).

It is important to document the existing paint system on a bridge. The paint type may be shown on the bridge drawings or specifications. Some agencies list the paint type and application date on the bridge. Once the existing paint is determined, a compatible paint for any required rehabilitation can be chosen.

Protection of Suspension Cables

Suspension cables of steel suspension bridges are particularly difficult to protect from corrosion. One method is to wrap the cables with a neoprene elastomeric cable wrap system or with a glass-fabric-reinforced plastic shell. In some cases, the elastomeric cable wrap has retained water and accelerated corrosion. Another method is to pour or inject paints into the spaces between the cable strands. Commonly, inhibitive pigments, such as zinc oxide, in an oil medium are used. Red lead pigment was commonly used in the past. Lead constitutes a significant health hazard, and care must be exercised care when inspecting cables. Do not inhale or ingest old paint. The paint on the exterior surface of a suspension cable dries, but the paint on the interior, surrounding individual strands, stays in the liquid, uncured state for years. The exterior of the cable is often topcoated with a different paint, such as an aluminum pigmented oil-based paint. Another option to protect suspension cables is to wrap tightly with small diameter wires. This allows the cable to "breath" while still providing a protective cover.

A newer technique used to resist the corrosion process of suspension cables is forced air dehumidification. On larger structures (such as the Kobe Bridge in Japan and the Ben Franklin Bridge in Pennsylvania), dry air is passed through the cables, which does not allow the steel to be exposed to moisture. For this protection system to work, the relative humidity of the forced air should be less than approximately 60%.

Weathering Steel

In the proper environments, weathering steel does not require painting but produces its own protective coating. When exposed to the atmosphere, weathering steel develops a protective oxide film, which seals and protects the steel from further corrosion. This oxide film is actually an intended layer of surface rust, which protects the member from further corrosion and loss of material thickness.

Weathering steel was first used for bridges in 1964 in Michigan. Since then, thousands of bridges have been constructed of weathering steel in the United States. The early successes of weathering steel in bridges led to the use of this steel in locations where the steel could not attain a protective oxide layer and where corrosion progressed beyond the intended layer of surface rust. Therefore, it is important for the inspector to distinguish between the protective layer of rust and advanced corrosion that can lead to section loss. It is also important to note that fatigue cracks can initiate in rust pitted areas of weathering steel.

The frequency of surface wetting and drying cycles determines the oxide film's texture and protective nature. The wetting cycle includes the accumulation of moisture from rainfall, dew, humidity, and fog, in addition to the spray of water from traffic. The drying cycle involves drying by sun and wind. Alternate cycles of wetting and drying are essential to the formation of the protective oxide coating. The protective film will not form if weathering steels remain wet for long periods of time.

Uses of Weathering Steel

Weathering steels may be unsuitable in the following environments:

- Areas with frequent high rainfall, high humidity, or persistent fog
- Marine coastal areas where the salt-laden air may deposit salt on the steel, which leads to moisture retention and corrosion
- Industrial areas where chemical fumes may drift directly onto the steel and cause corrosion
- Areas subject to "acid rain" which has a sulfuric acid component

The location and geometrics of a bridge also influence performance of weathering steel. Locations where weathering steel may be unsuitable include:

- Tunnel-like situations which permit concentrated salt-laden road sprays,

caused by high-speed traffic passing under the bridge, to accumulate on the superstructure
- Low level water crossings where insufficient clearance over bodies of water exists so that spray and condensation of water vapor result in prolonged periods of wetness

2.3.8 Inspection Procedures for Steel

There are three basic procedures used to inspect a steel member. Depending on the type of inspection, the inspector may be required to use only one individual procedure or all procedures. They include:

- Visual
- Physical
- Advanced inspection techniques

Visual Examination

Steel Members

Fatigue

Steel members should be inspected for corrosion, section loss, buckling, and cracking.

Some common inspection locations and signs of fatigue distress include:

- Bent or damaged members - determine the type of damage (e.g., collision, overload, or fire), measure the variance from proper alignment, and check for cracks, tears, and gouges near the damaged location
- Corrosion, which could reduce structural capacity through a decrease in member section and make the member less resistant to both repetitive and static stress conditions; since rust continually flakes off of a member, the severity of corrosion cannot always be determined by the amount of rust; therefore, corroded members must be examined by physical as well as visual means (see Figure 2.3.16)
- Fatigue cracks - fatigue cracks are common at certain locations on a bridge, and certain inspection procedures should be followed when fatigue cracks are observed (see Figure 2.3.17 and Topic 8.1 for additional information about fatigue cracks)
- Other stress-related cracks - determine the length, size, and location of the crack
- Points on the structure where a discontinuity or restraint is introduced
- Loose members which could force the member or other members to carry unequal or excessive stress
- Damaged members, regardless of damage magnitude, which are misaligned, bent, or torn
- Welded details
- Repairs that show indiscriminate welding or cutting procedures
- Areas of excessive vibrations or twisting

Inspection procedures for in-plane fatigue cracks:
- Report the fatigue crack immediately

- Determine the visual ends of the crack
- Examine other identical details on the bridge for cracks
- Examine other details for breaks in the paint and the formation of oxide (rust)
- If a suspect area is located, a more detailed examination, such as blast cleaning and using dye penetrant or ultrasonic testing, is required

Figure 2.3.16 Corrosion of Steel

Figure 2.3.17 Fatigue Crack

Protective Coatings

Knowing where to inspect is just as important as knowing how to inspect.

Areas to Inspect

Rust typically starts in a few characteristic places, then spreads to larger areas.

Examine sharp edges and square corners of structural members (see Figure 2.3.18). Paint is generally thinner at sharp edges and corners than at rounded edges and corners or flat surfaces. Rusting starts at sharp edges, then undercuts intact paint as it spreads away from the edge. Inside square corners often receive an extra thick layer of paint due to double or triple passes made over them. Extra thick layers are prone to cracking, exposing the steel. It is difficult to completely remove dirt and spent blast cleaning abrasive from inside corners. Painting over this foreign material results in early peeling and corrosion.

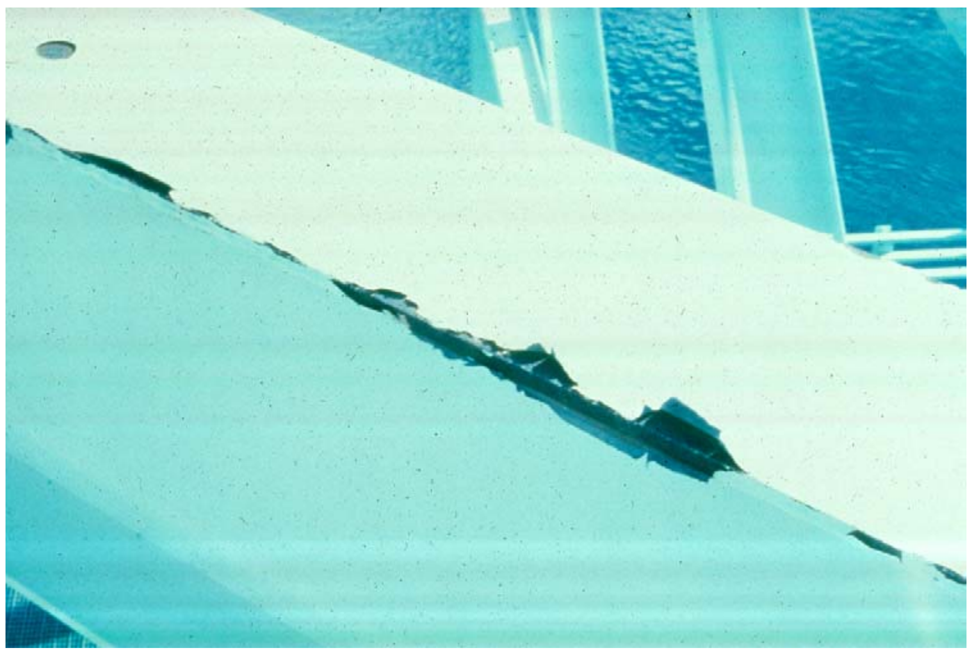


Figure 2.3.18 Edge Failure on Painted Steel Beam

Examine all areas that retain moisture and salt. Check under scuppers and beneath downspouts. Check horizontal surfaces under the edge of bridge decks and under expansion dams, where roadway deicing salt runoff collects (see Figure 2.3.19). Examine the bottom inside flange of girders.

Inspect inaccessible or hard-to-reach areas that may have been missed during painting. A flashlight and inspection mirror may be needed here. Examine the inside surfaces of lattice girders and beams. Examine the top surface of girder upper flanges under the bridge deck, if possible.

Inspect around bolts, rivets, and pins (see Figure 2.3.20). Rust detected around the heads may indicate corrosion along the entire length of the bolt, rivet, or pin, causing reduced structural integrity.

Examine roadway splash zones, where debris and corrosive deicing salt-laden water are directly deposited on painted members by passing traffic (see Figure 2.3.21). On through-truss bridges, this includes some bracing members above the roadway.

Examine areas exposed to wind and rain, seawater spray, and other adverse weather conditions.

Figure 2.3.19 Water and Salt Runoff Under Expansion Dam Deck Opening

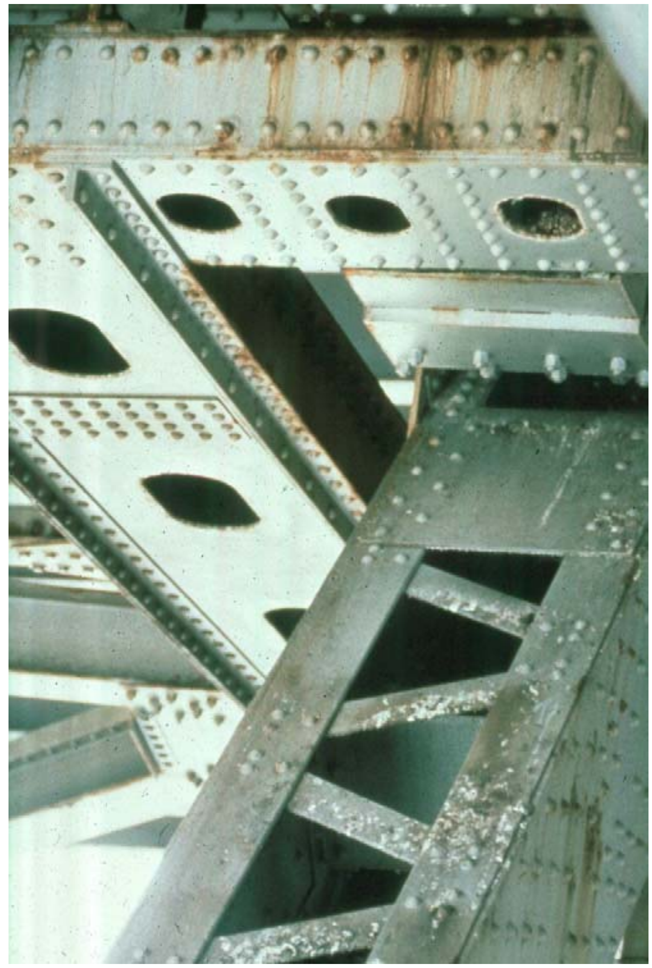


Figure 2.3.20 Corroding Rivet Head

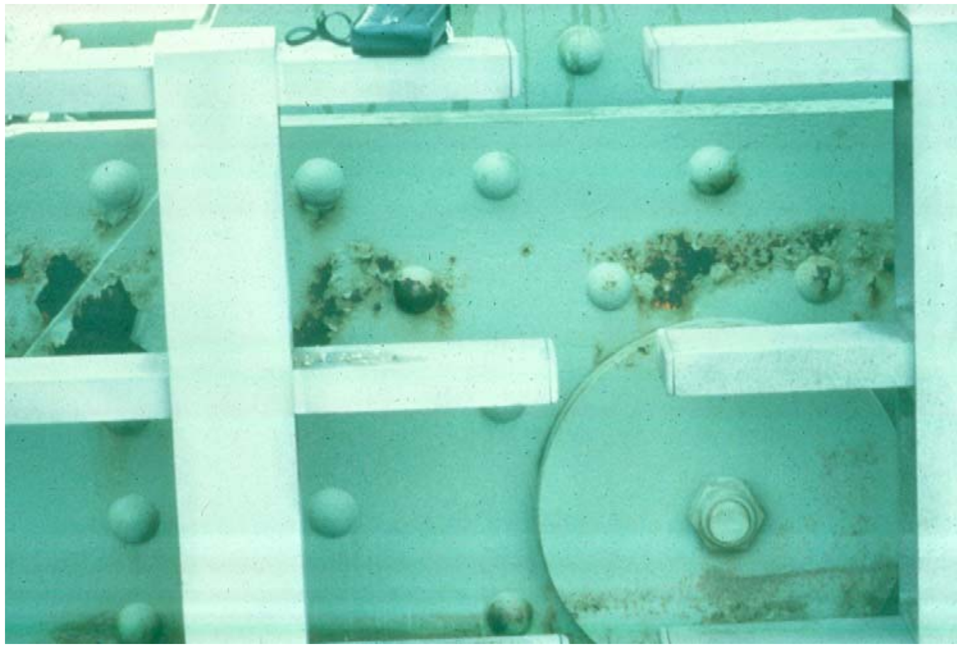


Figure 2.3.21 Roadway Splash Zone Damage (Note Aluminum Bridge Railing in Foreground)

Weathering Steel

It is particularly important for weathering steel to be inspected in the following locations:

- Where water ponds or the steel remains damp for long periods of time due to rain, condensation, leaky joints, or traffic spray
- Where debris is likely to accumulate
- Where the steel is exposed to salts and atmospheric pollutants
- Near defective joints or drainage devices

Color

The color of the surface of weathering steel is an indicator of the protective oxide film (see Figure 2.3.22). The color changes as the oxide film matures to a fully protective coating. Figures 2.3.23 through 2.3.26 correlate the color of the weathering steel and the degree of protection.

Figure 2.3.22 Color of Oxide Film is Critical in the Inspection of Weathering Steel; Dark Black Color in an Indication of Non-protective Oxide

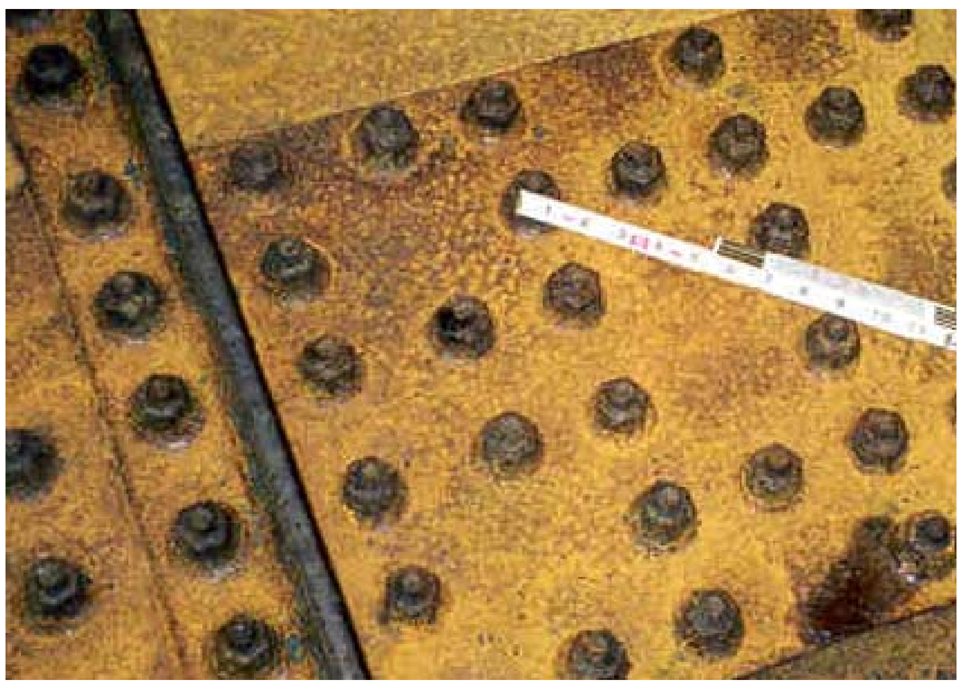


Figure 2.3.23 Yellow Orange – Early Stage of Exposure or Active Corrosion

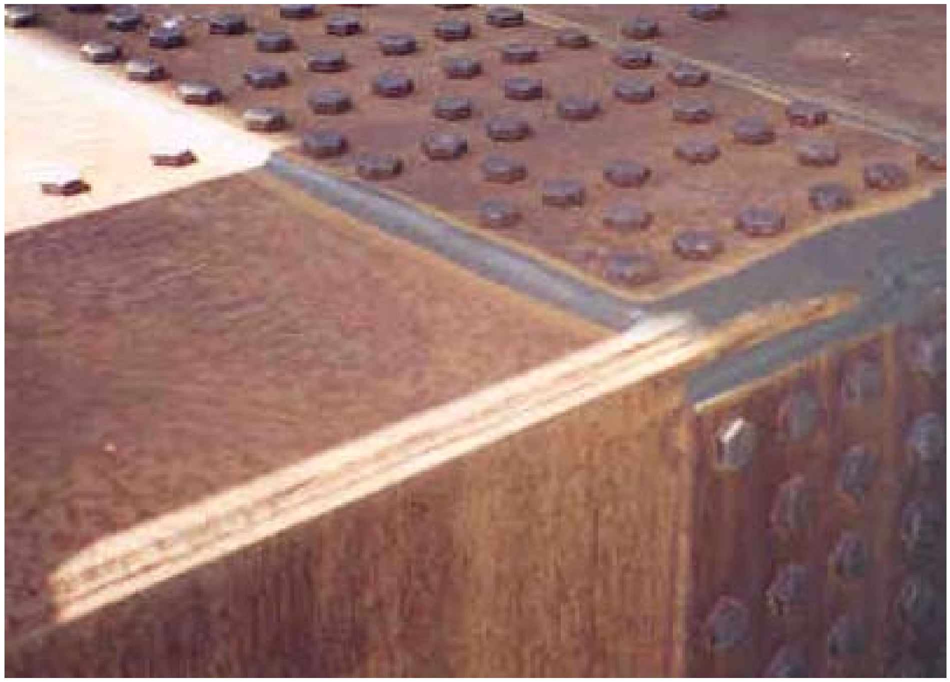


Figure 2.3.24 Light Brown – Early Stage of Exposure

Figure 2.3.25 Chocolate Brown to Purple Brown - Boldly Exposed and Good Degree of Protection

Figure 2.3.26 Black – Non-protective Oxide

An area of steel, which is a different color than the surrounding steel indicates a potential problem. The discolored area should be investigated to determine the cause of the discoloration. Color photographs are an ideal way to record the color of the weathering steel over time. A color coupon should be included in each photograph to enable comparison.

Texture

The texture of the oxide film also indicates the degree of protection of the film. An inspection of the surface by tapping with a hammer and vigorously brushing the surface with a wire brush determines the adhesion of the oxide film to the steel substrate. Surfaces, which have granules, flakes, or laminar sheets are examples of non-adhesion. Table 2.3.1 presents a correlation between the texture of the weathering steel and the degree of protection.

Appearance	Degree of Protection
Tightly adhered, capable of withstanding hammering or vigorous wire brushing	Protective oxide
Dusty	Early stages of exposure; should change after few years
Granular	Possible indication of problem, depending on length of exposure and location of member
Small flakes, 6 mm (1/4 inch) in diameter	Initial indication of non-protective oxide
Large flakes, 13 mm (1/2 inch) in diameter or greater	Non-protective oxide
Laminar sheets or nodules	Non-protective oxide, severe conditions

Table 2.3.1 Correlation Between Weathering Steel Texture and Condition

Physical Examination

Steel Members

Once the defects are identified visually, physical procedures must be used to verify the extent of the defect. For steel members, the main physical inspection procedures involve an inspection hammer and wire brush. Corrosion results in loss of member material. This partial loss of cross section due to corrosion is known as section loss. Section loss may be measured using a straight edge and a tape measure. However, a more exact method of measurement, such as calipers or a D-meter, should be used to measure the remaining section of steel. The inspector must remove all corrosion products (rust scale) prior to making measurements.

The inspector should measure the bridge members to verify that the sizes recorded in the plans or inspection report are accurate. If incorrect member sizes are used, then any load rating analysis for safe load capacity of the bridge is worthless.

Protective Coatings

The degree of coating corrosion must be assessed during the inspection. Coating corrosion is measured differently than structural corrosion. There are a variety of proprietary procedures which use a set of photographic standards to evaluate and categorize the degree and extent of coating corrosion on composite spans, cross frames, exterior fascias, and bearings. A simple method entails evaluation of painted surfaces in accordance with SSPC-Vis 2. Vis 2 is a pictorial standard for evaluating the degree of rusting on painted steel surfaces.

Mill Scale

Incomplete removal of mill scale can provide a starting point for corrosion. When mill scale cracks, it allows moisture and oxygen to reach the steel substrate. Mill scale accelerates corrosion of the substrate because of its electrochemical properties. To check for mill scale corrosion during a paint inspection, use a knife to remove a small patch of paint in random spots. Inspect the exposed surface for mill scale, either intact or rusted. Probe with a knife or other sharp object at weld spatter to check for rusting. Re-coat areas where paint is removed.

Invisible microscopic chloride deposits from deicing salt or seawater spray may permeate a corroding steel surface. Painting over a partially cleaned chloride-contaminated surface simply seals in the contaminant. Salt deposits draw moisture through the paint by osmosis, and corrosion will continue.

Paint Adhesion

Paint can undergo adhesion failure between paint layers or between the primer and steel. Some bridge painting contracts specify a minimum acceptable paint adhesion strength for new paint. Over time, however, adhesion strength may degrade as the paint weathers and is affected by sunlight, or as rusting occurs under the paint.

The simplest test of adhesion is to probe under paint with the point of a knife. A more quantitative evaluation is performed by a tape test, as described in Topic 2.1.

Paint Dry Film Thickness

There are a variety of instruments to measure the dry film thickness of paint applied to steel. Accuracy ranges from 10% +/- to 15% +/-, and they fall into three classes:

- Magnetic pull-off
- Fixed probe
- Destructive test

The magnetic pull-off dry film thickness gages use the attractive force between a magnet and the steel substrate to determine the paint thickness. The thicker the paint, the lower the magnetic force. These instruments must be calibrated prior to and during use with plastic shims of known thickness, or with ferrous plates coated with a non-ferrous layer. Such shims are produced by the National Bureau of Standards (NBS).

The fixed probe gages also use a magnet. Measurement of paint thickness is done by an electrical measurement of the interaction of the probe's magnetic field with the steel rather than by the force to move the magnet. They are normally calibrated with plastic shims. Neither the magnetic pull-off nor fixed probe gages can be used closer than one inch to edges, as this will distort the reading. SSPC-PA2 "Measurement of Dry Paint Thickness With Magnetic Gages" provides a detailed description of how to calibrate and take measurements using magnetic gages.

A destructive method for measuring dry film thickness uses the Tooke Gage described in Topic 2.1. An advantage of this method is that it can be used at any location, including close to edges. While the magnetic gages measure the combined thickness of all paint layers, the Tooke Gage measures each layer individually. Limitations of the destructive test are that only coatings up to 50 mils thick can be measured and multiple layers of the same color cannot be distinguished.

Repainting

If the coating is to be repainted, the type of in-place paint must be known, since different type paints may not adhere to each other. Methods described in Topic 2.1 can be used to determine the type of in-service paint.

Weathering Steel

Weathering steel with any of the following degree of protection should be inspected:

- Laminar texture of steel surface, such as slab rust or thin and fragile sheets of rust
- Granular and flaky rust texture of steel surface
- A very coarse texture
- Large granular (3 mm (1/8 inch) in diameter) texture
- Flakes (13 mm (1/2 inch) in diameter)
- Surface rubs off by hand or wire brush revealing a black substrate
- Surface is typically covered with deep pits

If such conditions are discovered, the following steps should be taken to determine the adequacy of the oxide film:

- Scrape the surface of the steel to the bare metal
- Check to determine the extent of pitting
- Measure the metal section loss with calipers or an ultrasonic thickness gauge

It is important to set a benchmark at the point where the metal thickness measurement is taken so that any metal loss may be monitored with future measurements. Benchmarks are important since steel rolled sections and steel plates often vary within acceptable tolerances in thickness from the nominal thickness values.

Data obtained from the inspection should include visual observations of the steel (e.g., color, texture, and flaking), physical measurements with a thickness gauge, and observation of environmental conditions.

Advanced Inspection Techniques

In addition, several advanced techniques are available for steel inspection. Nondestructive methods, described in Topic 13.3.2, include:

- Acoustic emissions testing
- Computer programs
- Computer tomography
- Corrosion sensors

- Dye penetrant
- Magnetic particle
- Radiographic testing
- Robotic inspection
- Ultrasonic testing
- Eddy current

Other methods, described in Topic 13.3.3, include:

- Brinell hardness test
- Charpy impact test
- Chemical analysis
- Tensile strength test

2.3.9 Other Bridge Materials

Cast Iron

Iron is an elemental metal smelted from iron ore. Cast iron is the most widely used cast metal. However, it is easily fractured by shocks and has low tensile strength due to a large percentage of free carbon and slag. Consequently, it is basically a poor bridge construction material and is not used in new bridge construction today. It may, however, be found in compression members of old bridges.

Cast iron is gray in color due to the presence of tiny flake-like particles of graphite (carbon) on the surface. It has a unit weight of about 7210 kg/m^3 (450 pcf).

Properties of Cast Iron

Some of the mechanical properties of cast iron include:

- Strength - tensile strength varies from 172 MPa (25,000 psi) to 345 MPa (50,000 psi), while compressive strength varies from 448 MPa (65,000 psi) to 1,035 MPa (150,000 psi)
- Elasticity - cast iron has an elastic modulus of 89,635 MPa (13,000,000 psi) to 206,850 MPa (30,000,000 psi): elasticity increases with a decrease in carbon content
- Workability - cast iron possesses good machinability, and casting is relatively easy and inexpensive
- Weldability - cast iron can not be effectively welded due to its high free carbon content
- Corrosion resistance - cast iron is generally more corrosion resistant than the other ferrous metals
- Brittleness - cast iron is very brittle and prone to fatigue-related failure when subjected to bending or tension stresses

Types of Cast Iron Deterioration

The primary forms of deterioration in cast iron are similar to those in steel (refer to Topic 2.3.4).

Wrought Iron

When iron is mechanically worked or rolled into a specific shape, it is classified as wrought iron. This process results in slag inclusions that are embedded between the microscopic grains of iron. It also results in a fibrous material with properties in the worked direction similar to steel. Wrought iron is no longer made in the United States. However, wrought iron tension members still exist on some older bridges, and it was well-suited for use in the early suspension bridges.

Properties of Wrought Iron

Some of the mechanical properties of wrought iron include:

- Strength - wrought iron is anisotropic (i.e., its strength varies with the orientation of its grain) due to the presence of slag inclusions; compressive strength is about 241 MPa (35,000 psi), while tensile strength varies between 248 MPa (36,000 psi) and 345 MPa (50,000 psi)
- Elasticity - modulus of elasticity ranges from 165,000 MPa (24,000,000 psi) to 200,000 MPa (29,000,000 psi), nearly as high as steel
- Impact resistance - wrought iron is tough and is noted for impact and shock resistance
- Workability - wrought iron possesses good machinability
- Weldability - wrought iron can be welded, but care should be exercised when welding the metal of an existing bridge
- Corrosion resistance - the fibrous nature of wrought iron produces a tight rust which is less likely to progress to flaking and scaling than is rust on carbon steel
- Ductility - wrought iron is generally ductile; reworking the wrought iron causes a finer and more thread-like distribution of the slag, thereby increasing ductility

Types of Wrought Iron Deterioration

The primary forms of deterioration in wrought iron are similar to those in steel (refer to Topic 2.3.4).

Aluminum

Aluminum is widely used for signs, light standards, railings, and sign structures. Aluminum is seldom used as a primary material in the construction of vehicular bridges.

Properties of Aluminum

The properties of aluminum are generally similar to those of steel (refer to Topic 2.3.3). However, a few notable differences exist:

- Weight - aluminum alloy has a unit weight of about 2800 kg/m^3 (175 pcf)
- Strength - aluminum is not as strong as steel, but alloying can increase its strength to that of steel

- Corrosion resistance - aluminum is highly resistant to atmospheric corrosion
- Workability - aluminum is easily fabricated, but welding of aluminum requires special procedures
- Durability - aluminum is durable
- Expense - aluminum is more expensive than steel

Types of Aluminum Deterioration

The primary forms of deterioration in aluminum are:

- Fatigue cracking - the combination of high stresses and vibration caused by wind produces fatigue
- Pitting - aluminum can pit slightly, but this condition rarely becomes serious

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Topic 2.4 Stone Masonry

2.4.1 Introduction

Stone masonry is seldom used in new bridge construction today except as facing or ornamentation. However, many old stone bridges are still in use and require inspections (see Figure 2.4.1). Granite, limestone, and sandstone are the most common types of stone that were used and are still seen today in bridges. In addition, many smaller bridges and culverts were built of locally available stone. Stone masonry typically has a unit weight of about 170 pcf.

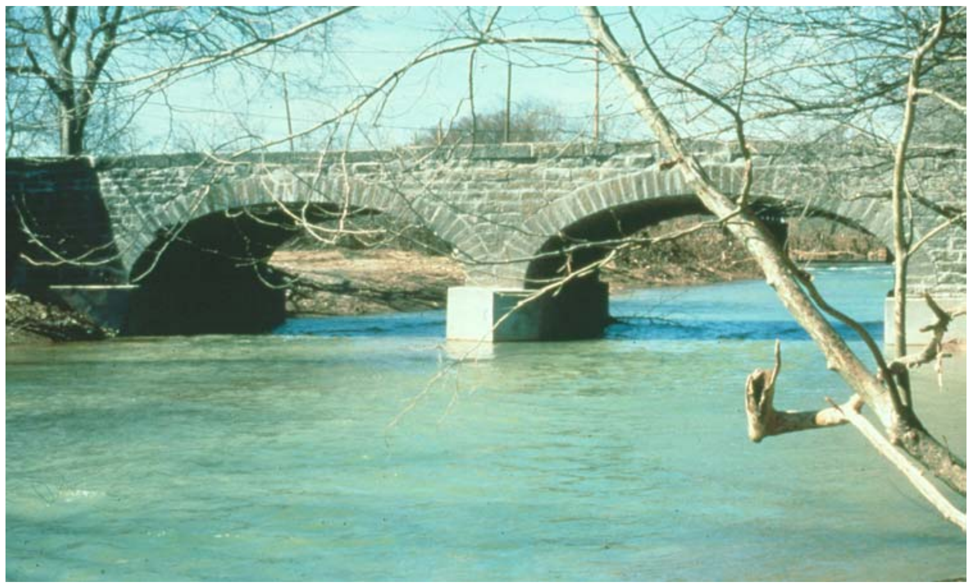

Figure 2.4.1 Stone Masonry Arch

2.4.2 Properties of Stone Masonry

The physical properties of stone masonry in bridge applications are of primary concern. Strength, hardness, workability, durability, and porosity properties of both the stone and the mortar play important roles in the usage of stone masonry.

Stone Masonry

Strength

Stone generally has sufficient strength to be used as a load-bearing bridge member, even though the strength of an individual stone type may vary tremendously. As an example, granite's compressive strength can vary from 7,700 to 60,000 psi (53 to 414 MPa). For the typical bridge application, a stone with a compressive strength of 5,000 psi (34.5 MPa) is acceptable. The mortar is almost always weaker than the stone.

Hardness

The hardness of stone varies based on the stone type. Some types of sandstone are soft enough to scratch easily, while other stones may be harder than some grades of steel.

Workability

Workability measures the amount of effort needed to cut or shape the stone. Harder stones are not as workable as softer stones.

Durability

The durability of stone depends on how well it can resist exposure to the elements, rain, wind, dust, frost action, heat, fire, and air-borne chemicals. Some stone types are so durable that they effectively resist the elements for two hundred years, while other stone types deteriorate after about ten years.

Porosity

Porosity in a stone indicates the amount of open or void space within that stone. All stone has some degree of porosity. Stone that is less porous can resist freeze/thaw action better than stone with a higher degree of porosity. Water absorption is directly related to the degree of porosity.

Mortar

Mortar is primarily composed of sand, cement, lime and water. The cement is generally Portland cement and provides strength and durability. Lime provides workability, water retentivity and elasticity. Sand is filler and contributes to economy and strength. The water, as in the case of concrete, can be almost any potable water. See Topic 2.2 for more information on mortar.

2.4.3 Stone Masonry Finishing Methods

There are three general methods of stone masonry construction:

- Rubble masonry
- Squared-stone masonry
- Ashlar Masonry

Rubble Masonry

Rubble masonry consists of rough stones which are un-squared and used as they come from the quarry. It could be constructed to approximate regular rows or courses (coursed rubble) or could be un-coursed (random rubble). Random rubble was the least expensive type of stone masonry construction and was considered strong and durable for small spans if well constructed.

Squared-Stone Masonry

Squared-stone masonry consists of stones, which are squared and dressed roughly. It could be laid randomly or in courses.

Ashlar Masonry

Ashlar consists of stones, which are precisely squared and finely dressed. Like squared-stone masonry, it could be laid randomly or in courses.

2.4.4 Types and Causes of Stone Masonry and Mortar Deterioration

The primary types of deterioration in stone masonry are:

- Weathering – hard surfaces degenerate into small granules, giving stones a smooth, rounded look; mortar disintegrates
- Spalling – small pieces of rock break out
- Splitting – seams or cracks open up in rocks, eventually breaking them into smaller pieces (see Figure 2.4.2)
- Fire – masonry is not flammable but can be damaged by high temperatures

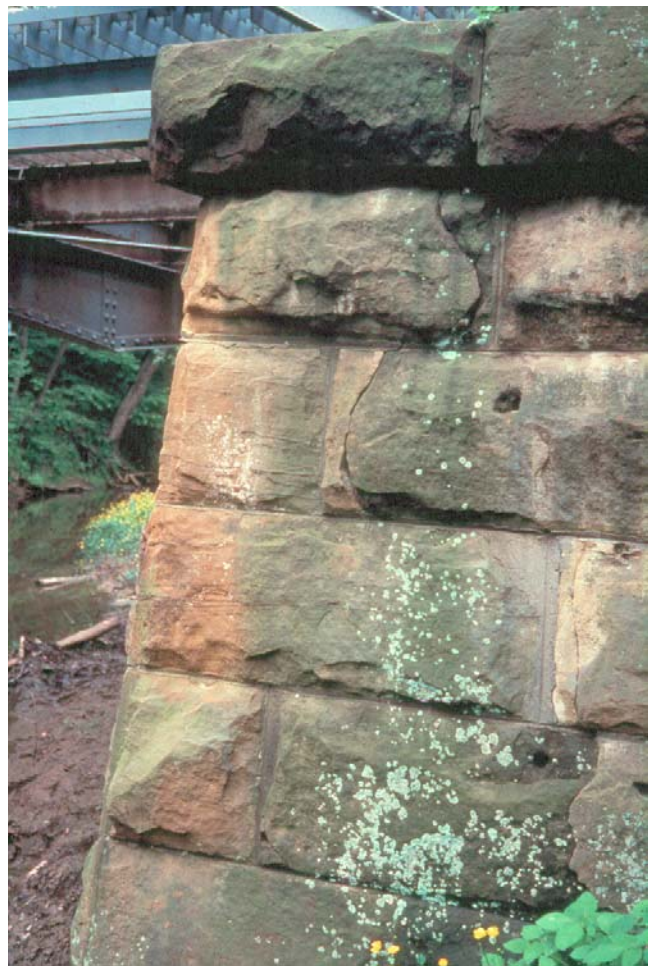

Figure 2.4.2 Splitting in Stone Masonry

Some of the major causes of these forms of deterioration are:

- Chemicals – gases and solids, such as deicing agents, dissolved in water often attack stone and mortar; oxidation and hydration of some compounds found in rock can also cause damage
- Volume changes – seasonal expansion and contraction can cause fractures to develop, weakening the stone

- Frost and freezing – water freezing in the seams and pores can spall or split stone or mortar
- Abrasion – due primarily to wind or waterborne particles
- Plant growth – roots and stems growing in crevices and joints can exert a wedging force, and lichen and ivy can chemically attack stone surfaces
- Marine growth – chemical secretions from rock-boring mollusks deteriorate stone

Two major factors that affect the durability of stone masonry include:

- The proper curing of mortar
- Correct placement of stones during construction

2.4.5 Protective Systems

The different types of protective systems used for concrete can also be used for stone masonry. The two most common systems that are used are paints and water repellant membranes or sealers. See Topic 2.2 for a complete description of the different types of protective systems.

2.4.6 Inspection Procedures for Stone Masonry and Mortar

The examination of stone masonry and mortar is similar to that of concrete. The joints should be carefully inspected for cracks and other forms of mortar deterioration. Inspection techniques are generally the same as for concrete. (See Topic 2.2 for the examination of concrete.)